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PROCEEDINGS

OF THE

TWENTY-FOURTH ANNUAL CONVENTION

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† Appointed in accordance with Article II, Sec. 4, By-Laws, to fill unexpired term of E. D. Boyer.

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CONTENTS

TECHNICAL COMMITTEES.

(For Convention Year February, 1928, to February, 1929.)

G-1.—Committee on Committees.

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G-7.—Program.

S. C. Hollister, *Chairman*.
Harvey Whipple, *Secretary*.
A. E. Lindau, R. J. Wig.

J-1.—Joint Committee on Concrete and Reinforced Concrete—Institute Representation.

S. C. Hollister, *Chairman*, Franklin Trust Bldg., Philadelphia.
A. B. McMillan, E. J. Moore, R. W. Lesley, H. D. Loring.

J-2.—INSTITUTE REPRESENTATION ON JOINT CULVERT PIPE COMMITTEE.

B. S. PEASE, *Chairman*, 208 So. LaSalle St., Chicago, Ill.
F. F. LONGLEY.

J-3.—TECHNICAL PROBLEMS IN CEMENT MANUFACTURE.

HENRY C. TURNER, *Chairman*, 420 Lexington Ave., New York City.
W. K. HATT, M. M. UPSON, RICHARD L. HUMPHREY.

J-4.—CEMENT.

E. D. BOYER, *Chairman*, 25 Broadway, New York City.
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C-4.—FORMS FOR CONCRETE BUILDING CONSTRUCTION.

E. C. HARDING, *Chairman,* Ferro Concrete Construction Co., Cincinnati, O.
D. B. NEIDERLANDER, J. A. TURNER, R. E. BATES, L. H. USILTON, W. J. CAMLIN, W. H. O'BRIEN.

C-6.—FIELD METHODS.

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P-6.—CONCRETE PRODUCTS PLANT OPERATION.

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AUSTIN CRABBS, A. G. SWANSON, C. E. LINDSLEY, EARLE D. MCKAY, GEORGE CHANDLER, C. L. DOUTHETT, C. L. BOURNE, O. L. BRADSHAW, EINAR CHRISTENSEN, C. J. HERZOG, C. H. HUNSACKER, MARTIN HAMMERSCHMIDT.

P-7.—CONCRETE PIPE, DRAIN TILE AND CONDUIT.

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T-2.—COLOR PIGMENTS IN CONCRETE.

RAYMOND WILSON, *Chairman*, 33 West Grand Ave., Chicago, Ill.

BY-LAWS.

AMERICAN CONCRETE INSTITUTE.

ARTICLE I.

MEMBERS.

SECTION 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

SEC. 2. A firm or company shall be treated as a single member.

SEC. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

SEC. 4. Application for membership shall be made to the Secretary on a form prescribed bythe Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

SEC. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

SEC. 6. The Board of Direction may confer honorary memberships in recognition of services of an extraordinarily meritorious character before the Institute. Honorary members shall be entitled to full membership privileges without the payment of dues.

ARTICLE II.

OFFICERS.

SECTION 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the

Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

SEC. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. There shall be a Committee of five members on Nomination of Officers elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter-ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election as President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election,

appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President on the first regular session of the Annual Convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days prior to the date of Convention.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence July 1st.

SEC. 2. The annual dues shall be twelve dollars and fifty cents ($12.50), payable annually in advance from the first of the month following notification of the applicant of his election by the Board of Direction.

SEC. 3. Each member shall be entitled to receive one copy of one volume of the *Proceedings* for each membership year and additional volumes at a price fixed by the Board of Direction.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

STANDARDS.

SECTION 1. Proposed new or revised Standard Specifications, Standard Practice, and Standard Definitions when approved by a majority voting in the committee in which they originate, shall be submitted, in the form adopted in the Standard Form of Standards, to the Secretary of the Institute sixty days prior to the opening of the Annual Convention at which they are to be presented. The Secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual *Proceedings*, next issued as Tentative Standards. At a subsequent Annual Convention, they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

ARTICLE VI.

AMENDMENTS.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF PROCEEDINGS OF THE TWENTY-FOURTH ANNUAL CONVENTION.

Registration began at 9 A. M., February 28, and progressed so rapidly that 238 sat down to the noonday get-together luncheon.

FIRST SESSION, TUESDAY, 2 P. M., FERBUARY 28—WORKABILITY.

President M. M. Upson in the chair.

The following papers were presented: "A Study of Some Methods of Measuring Workability of Concrete" (Preprinted), by George A. Smith and George Conahey, Research Associates, Celite Company Fellowship, U. S. Bureau of Standards.

"The Cement Factor in Workability," by P. H. Bates, U. S. Bureau of Standards.

"Gradation and Character of Aggregates as a Factor in Workability" (Preprinted), by A. T. Goldbeck, Director of Research, National Crushed Stone Association.

"Water as a Factor in Workability" (Preprinted), by R. L. Bertin, The White Construction Co., New York City.

"Workability Means Durability to the Engineer" (Preprinted), by R. W. Atwater, in charge of structural engineering, McClellan and Junkersfeld, New York City.

"What Workability Means to the Contractor," by Nelson L. Doe, general superintendent, Turner Construction Co., New York City.

Discussion was all reserved until the six papers of the symposium had been presented. More than 400 attended this first session—both attendance and discussion showing a lively interest in the subject.

SECOND SESSION, TUESDAY, 8 P. M., FEBRUARY 28—ARCHITECTURE.

President Upson in the chair.

A paper "Methods of Decorative Painting on Concrete Surfaces," by Sidney F. Ross, of Arnold W. Brunner Associates, Architects, New York City, was presented in Mr. Ross' absence by Virgil L. Johnson, architect, Philadelphia.

Two other papers were presented by their authors—both papers being well illustrated by stereopticon slides: "Reinforced Concrete as Applied to

Monumental Types of Buildings," by Emil Praeger, engineer of Bertram Grosvenor Goodhue Associates, architects, New York City; and "Reinforced-Concrete Walls for Buildings," by W. E. Hart, manager, Structural and Technical Bureau, Portland Cement Association, Chicago.

Arthur J. Meigs, architect, Philadelphia, presented illustrated discussion of Mr. Praeger's paper.

Mr. Hart's paper also evoked considerable discussion, including advance written discussion by John I. Bright and Virgil L. Johnson, both architects of Philadelphia.

THIRD SESSION, WEDNESDAY, 9.30 A. M., FEBRUARY 29—RESEARCH.

Past-President A. E. Lindau in the chair.

The report of Committee E-3, Research, H. F. Gonnerman, chairman, having been preprinted, was presented by title only by F. E. Richart, secretary, in Mr. Gonnerman's absence, and was received for publication. The report consists chiefly of a compilation of references to investigations under way, and to reports published within the year.

The report of Committee E-5, Aggregates, R. W. Crum, chairman, having been preprinted, was presented by title only by F. H. Jackson, secretary, in Mr. Crum's absence. The report consists of a discussion of the development of an abrasion test for gravel and a tentative method for such a test. The report was accepted for publication.

The following papers were presented and offered for discussion:

"A Method for Predicting Concrete Strength with Increased Precision" (Preprinted), by Herbert J. Gilkey, associate professor of civil engineering, University of Colorado. (Presented by title in the author's absence.)

"Notes on the Progress of Some Studies of the Crazing of Portland Cement Mortars" (Preprinted), by P. H. Bates and C. H. Jumper, U. S. Bureau of Standards. (Presented by Mr. Bates.)

"Crazing in Concrete and the Growth of Hair Cracks into Structural Cracks" (Preprinted), by Alfred H. White, professor of chemical engineering, and Vilhelm A. Aagaard and Axel O. L. Christensen, of the department of Chemical engineering, University of Michigan. (Presented by Professor White.)

"The Carrying Capacity of Semi-Circular Hooks" (Preprinted), by T. D. Mylrea, professor of building construction, Carnegie Institute of Technology, Pittsburgh.

"Some Suggestions from the Results of the Stevenson Creek Arch Dam Investigation," by W. A. Slater, U. S. Bureau of Standards.

"Flow of Concrete Under Sustained Compressive Stress" (Preprinted), by Raymond E. Davis, professor of civil engineering, University of California.

J. H. Chubb and C. E. Lindsley were appointed tellers to count the annual ballots for officers.

FOURTH SESSION, WEDNESDAY, 2 P. M., FEBRUARY 29—CONCRETE STONE AND SPECIAL PRODUCTS.

Vice-President Boyer in the chair.

The following papers were presented:

"Concrete Roofing Tile Problems," by Leslie H. Allen, president, Hawthorne Roofing Tile Co., of N. Y., Inc. This was followed by a discussion of colors by Maximilian Toch.

"Viewpoint of the Architect and Engineer Regarding Concrete Masonry Units," by George J. Eyrick, Jr., Smith, Hinchman & Grylls, architects and engineers, Detroit. In Mr. Eyrick's absence, his paper was presented by Benjamin F. Wilk.

"Specifications for Concrete Stone" (Preprinted), by C. Van de Bogart, president, Economy Concrete Co., New Haven, Conn.

"The Need of a National Certification Plan," by M. A. Arnold, president, Arnold Stone, Brick & Tile Co., Inc., Jacksonville, Fla.

"Pacific Stone," by Gilbert E. Tucker, vice-president and manager, Pacific Stone Co., Seattle, Wash. Due to the author's unanticipated absence, his paper was presented only by title.

Following these papers, there was general discussion of the desirability of concrete stone specifications and of the adoption of a name for the product.

A resolution was adopted requesting the Board of Direction to appoint a committee of concrete stone manufacturers and others to formulate specifications under which a standard product may be produced and sold.

A paper, "Better Stucco," by William S. Steele, president, Mohawk Stucco Co., Inc., Brooklyn, N. Y., with discussion, concluded the session.

FIFTH SESSION, WEDNESDAY, 8 P. M., FEBRUARY 29—REINFORCED-CONCRETE DESIGN.

Past-President A. E. Lindau in the chair.

The following papers were presented:

"Skew Arch Design and Construction" (Preprinted), by S. C. Hollister, consulting engineer, Philadelphia.

"The Calculation of Flat Plates by the Elastic Web Method" (Preprinted), by Joseph A. Wise, assistant professor of structural engineering, University of Minnesota. (In the author's absence the paper was formally presented—but not read—by Louis Clousing.)

"Design and Cost Data for Proposed 1928 A. C. I. Building Code" (Preprinted), by A. R. Lord, consulting engineer, Chicago.

The report of Committee E-1, Reinforced-Concrete Building Design and specifications consisting of "Proposed Standard Building Regulations for Reinforced Concrete" (Preprinted), was presented by F. R. McMillan, chairman, who then offered a considerable list of amendments—nearly 50. Many of them involved discussion; many did not. All the amendments

proposed by the committee prevailed, some after modification from the floor. The session was a memorable one, lasting until 12.30 A. M.

SIXTH SESSION, THURSDAY, 9.30 A. M., MARCH 1—STANDARD CONCRETE BUILDING UNITS.

Past-President L. C. Wason in the chair.

The following papers were presented:

"How a State Law Helped Concrete Units in Wisconsin," by D. R. Collins, Milwaukee.

"Experience in the Use of Light-Weight Aggregate for Masonry Units," by A. W. Scheer, president, Best Block Co., Milwaukee.

There was detailed discussion tending to establish the fallacy of the absorption test and to introduce the importance of the so-called "wetness factor" by which porous light-weight aggregate shows high weather resistance.

"Drying Concrete Brick to Take Out the Shrinkage," by L. E. Grube, Sheboygan, Wisc.

The report of Committee P-1, Standard Concrete Building Units (Preprinted) covered these subjects: (1) Specifications for Concrete Brick; (2) Tests of Feed of Mix to Brick and Block Machines in Relation to Tamping; (3) Concrete Block and Tile Specifications; (4) Specifications for Concrete Manhole and Catch Basin Block; (5) Progress of Simplified Practice in Standardization of Sizes.

The first action of the convention was to adopt the committee's recommendations, with respect to Nos. 2, 3 and 5, receiving Nos. 2 and 5 for publication and adopting revisions in the Institute's Tentative Specifications for Concrete Block and Concrete Building Tile (P-1-A-26T) with respect to compression tests on different classes of block and tile—the revised specifications becoming P-1-A-28T.

The Tentative Specifications for Concrete Brick P-1-B-26T were continued as tentative without revision.

The Tentative Specifications for Concrete Manhole and Catch Basin Block, P-1-C-27T were continued as tentative for another year, with an amended title as follows: Concrete Sewer Manhole and Catch Basin Block P-1-C-28T.

SEVENTH SESSION, THURSDAY, 2 P. M., MARCH 1—BUSINESS—FIELD METHODS.

President Upson in the chair.

The report of the tellers, J. H. Chubb, chairman, showed ballots cast for officers and directors as follows:

President, one year, E. D. Boyer, 671 votes.

Vice-President, two years, D. A. Abrams, 677 votes.

Treasurer, one year, Harvey Whipple, 672 votes.

Director, First District, two years, A. C. Tozzer, 639 votes.

Director, Second District, two years, J. G. Ahlers, 644 votes.

Director, Sixth District, two years, R. J. Wig, 638 votes.

(At a meeting of the Board of Direction, in accordance with the By-Laws, Article II, Sec. 4, S. C. Hollister, senior director, was appointed vice-president to fill the unexpired term in that office of E. D. Boyer. J. C. Pearson was appointed director to fill the unexpired term of S. C. Hollister, third district. Harvey Whipple was reappointed secretary.)

Mr. Upson presented the new president.

Following a few words of acknowledgment by Mr. Boyer, President Upson expressed appreciation of the work of the Philadelphia Activities Committee toward the success of the convention, and the management of the Benjamin Franklin for courtesy and co-operation.

Mr. Upson expressed regret at the loss of Frank Wight, and asked the convention to rise out of respect to his memory.

The Secretary presented Honor Roll prizes for the ten men who sponsored the most new members in the year ended February 1, 1928:

First—W. C. Voss, Boston—30 members; second—D. A. Abrams, New York—23 members.

Third—John G. Ahlers, New York—12 members; fourth—H. B. Emerson, Chicago—12 members; fifth—W. E. McComas, Philadelphia—9 members; sixth—Miguel Villa, Cuba—9 members; seventh—H. F. Gonnerman, Chicago—9 members; eighth—I. E. Burks, Canada—8 members; ninth—W. R. Harris, Milwaukee—8 members; tenth—F. R. McMillan, Chicago—8 members.

At the conclusion of the Business Session, Past-President Wason took the chair. Mr. Wason, president of the Institute for two terms, 1915 and 1916, called attention to the operation of the By-Laws, which removed him from the Board of Direction after a service of 20 years. The five last living past-presidents, who continue membership in the Institute, are members of the Board. Thus, two years ago, when Mr. Lindau was succeeded by Mr. Upson as president, Richard L. Humphrey, first president of the Institute, passed off the Board. This year, on Mr. Upson's becoming a past-president, Mr. Wason, the second president, completes his Board service.

The report of Committee S-6, Concrete Roads and Pavements, W. M. Acheson, chairman, L. S. Trainor, secretary, had been preprinted. The report carried recommendations of minor changes in existing specifications:

First—To submit to letter ballot of Institute membership minor tentative revisions adopted in 1925, as Standard Specifications for One-Course Portland Cement Concrete Pavement for Highways (S-6-A-24), and three other specifications of which it is a part. S-6-B-25, S-6-C-25 and S-6-D-25. and S-6-D-25. (The road specifications have since been approved by letter ballot.)

Second—To adopt tentatively a revision as preprinted, in the wording of Part II, Materials, Sec. E, Joint Filler, paragraph 18 of the specifications, S-6-A-24—affecting also specifications S-6-B-25, S-6-C-25 and S-6-D-25.

The report of the committee was adopted.

The paper by C. A. Wiepking, "Study of a Method for Testing Concrete in the Field" (preprinted), scheduled for the third session but carried over, was then presented. Written discussion by Professor Gilkey was presented by title in the author's absence.

Nathan C. Johnson, consulting engineer, New York City, presented his paper, "Better Concrete--Do We Mean It?" The manuscript having been prepared for publication, Mr. Johnson talked extemporaneously, enlarged upon his thought. Discussion followed.

Charles E. Covell's paper, "Heavy Duty Concrete Floors," having been preprinted, was presented in abstract by E. F. Archibald, also of the Austin Company, in Mr. Covell's absence, and a discussion of floor finishes followed.

"Experience with a Strength Specification Contract" (preprinted) was presented, in abstract only, by the authors, R. C. Johnson, Immel Construction Co., Fond du Lac, Wisc.

F. R. McMillan, whose "Concrete Primer" will be reprinted by the Institute for the distribution of thousands of copies at a nominal price, briefly outlined the purpose of the unique paper, in which it is attempted to set forth established knowledge of concrete and the improved methods in making it, in 142 questions and answers, logically arranged and briefly set forth within the compass of about 30 printed pages.

The convention closed with the dinner at 7 P. M., March 1, attended by about 200. Mr. Upson, the retiring president, made a brief address, in which he sketched the imposing progress of knowledge of and technique in concrete work, and the economic contribution in this advance in which the American Concrete Institute has had an increasingly important part.

President Toastmaster Upson then presented to Arthur R. Lord, the Wason medal for the most meritorious paper of the 1927 convention—"Notes on Concrete—Wacker Drive, Chicago," and the new Henry C. Turner medal of gold, founded for presentation not oftener than once each year, for "notable achievement in or service to the concrete industry," to Dr. Arthur N. Talbot, for "outstanding contributions to the knowledge of reinforced-concrete design and construction."

In his response, Dr. Talbot sketched briefly his part in studies of concrete in the last fifty years, and asking his audience to forget for a moment the present medal recipient, pointed out that bestowing such honors is praiseworthy, in giving proper recognition for service, because of the influence upon others than the medalist, in helping to develop and strengthen professional consciousness.

Mr. Upson then introduced Dr. John A. Miller, professor of astronomy, Swarthmore College, who made the evening's address on his chosen subject, centering on preparation for and observation of an eclipse of the sun in Sumatra.

AWARDS.

THE WASON MEDAL.

Awarded Each Year to the Author of the Most Meritorious Paper Presented to the Previous Annual Convention.

Awarded, 1928, to

Arthur R. Lord, for paper, "Notes on Concrete—Wacker Drive, Chicago," presented to the 1927 Convention.

Previous Wason Medal Awards.

1916 Convention Paper—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. Hull, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. Westergaard, "Moments and Stresses in Slabs."

1922 Convention Paper—George E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gages."

1923 Convention Paper—J. J. Earley, "Building the Fountain of Time."

1924 Convention Paper—Richard L. Humphrey, for two papers, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Convention Paper—E. A. Dockstader, for paper, "Reports of Tests Made to Determine Temperature in Reinforced-Concrete Chimney Shells."

1926 Convention Paper—A. Burton Cohen, for paper, "Correlated Considerations in the Design and Construction of Concrete Bridges."

THE HENRY C. TURNER MEDAL.

Presented Not Oftener Than Once Each Year, for "Notable Achievement in or Service to the Concrete Industry."

Awarded, 1928, to

Dr. Arthur N. Talbot, for "outstanding contributions to the knowledge of reinforced-concrete design and construction."

President's Address.

By M. M. Upson.*

Custom decrees that at each convention some statement be forthcoming from your president. Whether this is a penalty inflicted by the Institute upon itself or by a chief executive upon himself, deponent sayeth not. It is expected that I shall spread before you a detailed résumé of the unprecedented records of the past year, that you may be shown the pinnacles of 1927 achievements, and that you may be taken up to great heights from which the future of the industry may be viewed. To proceed upon this program I hesitate. My reason for my reluctance may be illustrated by an incident I experienced in California. I am sure you who hail from this wonderful golden state, which I so truly admire, will forgive a betrayal of one of its very few eccentricities.

I attended as a guest one of the celebrated Chamber of Commerce luncheons in the fair city of Los Angeles. It was held in the then new, beautiful and impressive Los Angeles Biltmore Hotel. I was told that there were twelve hundred people in attendance—ladies and gentlemen—seated in the spacious and highly decorated ballroom. It was an unusual sight—an inspiring spectacle. The president arose and addressed the meeting somewhat as follows: "Ladies and Gentlemen, Members and Guests of the Chamber of Commerce of Los Angeles. You are assembled here today—the largest, the greatest Chamber of Commerce in all the world—here in this magnificent hotel, the most beautiful hotel edifice not only in this country but in all the world—here on this glorious day, this perfect California day, where we are blessed with the most wonderful weather in all the wide world. And while we are assembled it is fitting that I should tell you another record—one of our members has today paid his dues to this Chamber of Commerce for fifty years in advance,—an unprecedented accomplishment, a record before all the world!"

Imitative of our enthusiastic Californian, we can say with perfect truthfulness that we are here today assembled in the greatest convention that the American Concrete Institute has ever held.

It is fitting that we should step aside in our mad rush and pause to watch this procession of achievements as it passes by. The progress is rapid and sure, but unless we stop to measure the distance traveled we little appreciate how far we have come.

In 1904 we met to discuss machinery mixers, handling devices, forms, and the like. Today we use a vocabulary then unknown—workability, fine-

* President, American Concrete Institute.

ness modulus, water-cement ratio, inundation, etc., etc. They all constitute milestones to mark advancement, and seemingly they appear more and more frequently during recent years.

It is but yesterday that we learned to predetermine with pencil and paper the strength of a mixture given the physical characteristics of the aggregate. And it has been even later that we have been able to establish a control that gives a concrete of uniform strength and characteristics. We are blossoming from the bud of guesswork to the full and glorious bloom of scientific certainty.

With each development comes a new field of inquiry which promotes increased productivity. They all lead to two great goals; namely, better product and lesser cost.

The American Concrete Institute aims, first, to assemble these facts as they develop; and second, to pass them on to the concrete-using world. We gather and we sow. Education is as important as is development. What gain to the industry, if a favored few hide their bits of truth, and the great outside world sails blindly on ever encountering uncharted rocks, courting disaster after disaster?

The confidence of the public comes from satisfactory performance. One failure vitiates a dozen successes. The weak link determines the strength of the chain. For this reason your board of direction is endeavoring each year to extend the influence of the Institute. We were less than five hundred in 1920—we are now more than twenty-five hundred strong.

We find that we must secure and print longer and more expensive papers; that we must communicate more frequently and more intimately with our members. All of this costs money, and for that reason last year we increased our dues. The increase is small and we believe is justified by the accomplishments.

Consider the contributions to the industry that have been forthcoming in this convention. I shall point to but a few of the high lights:

Let your minds dwell for a moment on the influence upon the quality of the concrete work of the world if the Concrete Primer which Mr. McMillan has so ably assembled can be placed in the hands of our construction superintendents and foremen. A new light and a better understanding of old facts will guide them away from many pitfalls.

So, too, will Mr. Lord's paper and the E-1 report constitute a new bible of designing facts, of inestimable value to concrete engineering. These are illustrative of the milestones that make and guide our forward progress.

The co-operative effort of the cement manufacturer has also greatly aided. Early strength cement solves problems of construction and cost which have hitherto baffled our greatest skill. Better quality and more uniform quality of cement make possible increased economy and scientific control. Light aggregates are now approaching with a promise that they will open to cement, doors that heretofore have been completely closed.

It is astounding to observe how this young stranger (Mr. Portland

Cement), who first came to our shores from foreign lands only a little more than a quarter of a century ago, has found his way into the homes and hearts of our best construction families. First he was received only by retaining walls, foundations and dams. Then, that brilliant amalgamation with the steel family provided him entrée to seemingly every construction aristocracy: buildings, bridges, roads, docks, bulkheads, pipe, piles, bins, tanks and even ships. One by one the lords of construction have opened their doors to this young stripling, until his popularity has carried the industry off its feet.

Of the $6,000,000,000 spent last year in construction, almost $2,000,000,000 represent structures in which concrete is the dominant material. In other words, one-third of the money spent in building goes into structures which are essentially built of cement in one form or another. If all this can be accomplished in a little more than a quarter of a century, with no precedent or scientific data for a foundation, what, with our present knowledge, may be expected within the next 25 years?

Two years ago we stood aghast at a 300-ft. reinforced-concrete arch. Today we watch with equanimity the erection of a span covering 650 ft. Who dares to measure the accomplishments of the future?

As light aggregate finds its way into the industry in commercial quantities and at lower cost, many of our present limitations in design and construction will disappear. Again the avenues of advance open, and who can say how far they will lead. This is but one of a dozen new developments which are creeping into the industry. Our eyes are blurred and bewildered as we try to gaze into the future. The world lies at our feet!

Our able secretary has very aptly stated the aim of our organization: "To provide a comradeship in finding the best ways to do concrete work of all kinds, and in spreading that knowledge."

Here lies the strength of our effort—a society numbering 450 eight years ago—2,500 today, and possibly and probably 5,000 five years hence—each member working shoulder to shoulder in earnest and honest endeavor to find co-operative ways of helping the cement world.

All great commercial and industrial attainments are based on this principle. This has been the source of America's unprecedented industrial success. Men have assembled in friendliness and a spirit of co-operation to solve great problems. They have been willing to sacrifice personal opinions and ambitions on the altar of tolerance and scientific advance. No problem can withhold its secrets against such tactics. No community can withstand the sales propaganda thus initiated. By such methods all avenues shall be open to our bidding.

A Study of Some Methods of Measuring Workability of Concrete.*

By George A. Smith and George Conahey.†

The workability of a concrete is that property which is indicated by the effort required to place the concrete in order to obtain a uniform and homogeneous finished product. This property was so defined by Messrs. Pearson and Hitchcock, Talbot and Richart, and other investigators. (Ref. 1, 2, 7, 8, 9, 10, 11, 12, 13.) Because workability is the resultant of several properties of the ingredients and not a simple physical property it is difficult of precise definition and measurement. It is affected by the proportions of the mix, the type and gradation of the aggregates, the cement and admixtures used, the quantity of mixing water, and possibly by other variables.

The terms "consistency" and "workability" as applied to concrete, are recognized as representing entirely different properties. Many investigators have clearly pointed out the difference between the terms. (Ref. 7, 8, 9, 10, 11, 12, 13.) Consistency has been defined as a property dependent upon the amount of water used in a particular mix whereas workability depends not only upon the water content but also upon the character and proportions of the materials used. In some of the early reports (Ref. 3, 4, 6) of tests of concrete, the word "consistency" has referred to the water content of the mix and the word "workable" used as a qualifying adjective describing the concrete. Later reports have used the term "workability" to define a property of the concrete.

Numerous devices have been developed for measuring the flowing and working properties of concrete and with a few exceptions the property measured has been more nearly the consistency than the workability. Among the various methods are the slump test (Ref. 14, 15, 16, 17, 18, 19, 20), the flow test (Ref. 22, 23, 24), the plate test (Ref. 10), the drop test (Ref. 23), and the penetration test (Ref. 8, 9).

All these devices and many new ideas were investigated before beginning these tests. It is evident that the slump cone cannot be used as a measure of workability because the same slump can be obtained with very rich smooth working mixtures and very lean harsh working mixtures by simply varying the water content. This same discussion is true of the flow table test, the drop test, the plate test, and of many other ideas suggested as methods for measuring workability. However, if the correct proportions for the desired workability are determined in the laboratory by an accepted workability measuring apparatus, the slump test might be used as a satisfactory test for field control.

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* Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

† Research Associates: The Celite Company Fellowship at the National Bureau of Standards.

Every device suggested for measuring workability was investigated. The penetration test as developed by Pearson and Hitchcock (Ref. 8, 9)

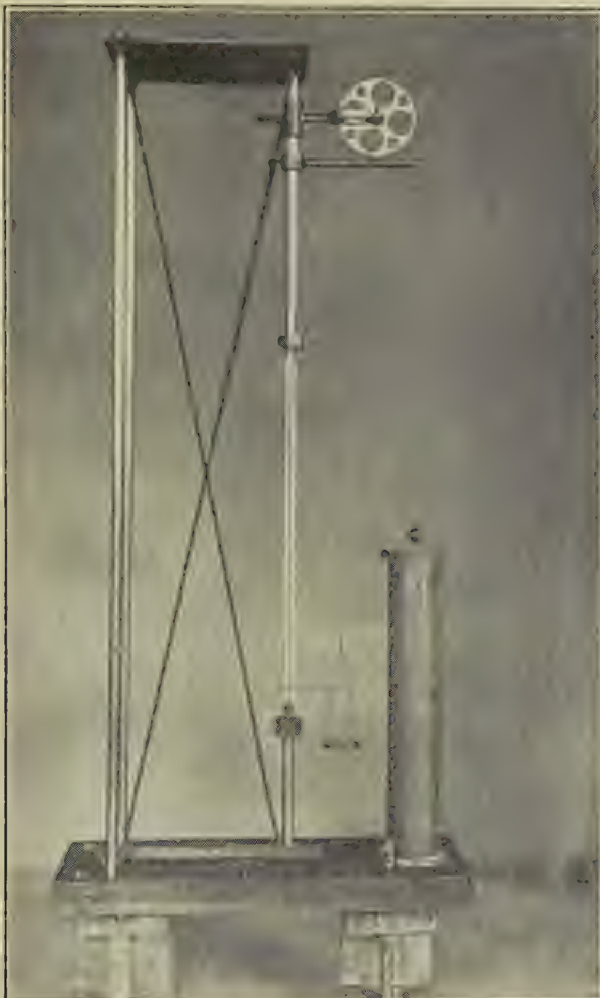

FIG. 1.—APPARATUS DEVELOPED TO STUDY THE PLASTIC PROPERTIES OF NEAT CEMENT PASTES.

was the only apparatus thus far developed that measured workability or at least measured a property different from that measured by the slump and flow tests. The work of Prof. A. N. Talbot (Ref. 26) in attempting

to deform a mass of concrete suggested a method which needed further investigation. The lubrication of a concrete mixture is one of the important factors affecting the workability. It was thought that a method which would measure the plastic properties of a neat cement paste might give information about the workability of the concrete made with that cement. It was suggested that a modification in the method of operating the flow table or modifications in the flow table apparatus might develop it into a method for measuring workability. Accordingly the following four methods were studied:

(1) Studies were made of methods for measuring the plastic properties of neat cement pastes.

(2) Investigations of modified forms of the flow table were carried out.

(3) Further investigation was made of the principle of mass deformation which was suggested by Prof. A. N. Talbot's early work in this field.

(4) A further study was made with Pearson and Hitchcock's penetration apparatus.

Study of the Plastic Properties of Cement Pastes.—The apparatus shown in Fig. 1 was developed from a suggestion of Dr. W. H. Herschel of the Bureau for the study of the plastic properties of neat cement pastes. The small ball was pulled through a column of the cement paste in the brass cylinder by means of weights in the scale pan. Data to show the relation of load to rate of shear were obtained. Although results which were of interest in their applicability to cements were secured, the work was discontinued because the apparatus could not be adapted to the measurement of the workability of concrete mixtures.

In this study of the problem it was learned that the addition of fine and coarse aggregates to a cement paste changes its nature. A moderately wet rich mix (1: 1: 2 mix, 3 to 4-in. slump) acts as a plastic material, that is, it will stand up under small initial loads but will flow under pressure. A lean mix (1: 2½: 5 mix) has more the properties of a granular material, that is, it bulks or swells when dampened, it settles into the closest packed condition when inundated and shows dilatancy effects or expands if the shape of the mass is changed when the material is in the closest packed condition.

The Study of Concrete on the Flow Table.—The same flow can be obtained by a change in the water content of very rich workable mixes and very lean non-workable mixes by the present method of operating the flow table. Because of this fact, this method of making the flow test is not a measure of the workability of the concrete. A study was made of modifications in the method of operating the flow table, in the shape of the table top and in the method of measuring the distribution of the concrete after making the test, which might make it possible to measure the workability on the flow table. The segregation or the separation of the lubricating medium from the aggregates is one of the causes for the lack of workability in a concrete. It appeared possible to measure the segregation by a proper

division of the concrete mass after making the flow test. This measurement was attempted by dividing the area of the flow table covered by the concrete into two equal parts by means of circular cutters. The flow table and cutters are shown in Fig. 2.

In this study a series of tests was made using the flat top table with a ½-in. drop, a second series with a ⅛-in. drop and a third series with a 1/16-in. drop. The shape of the top of the flow table was also changed

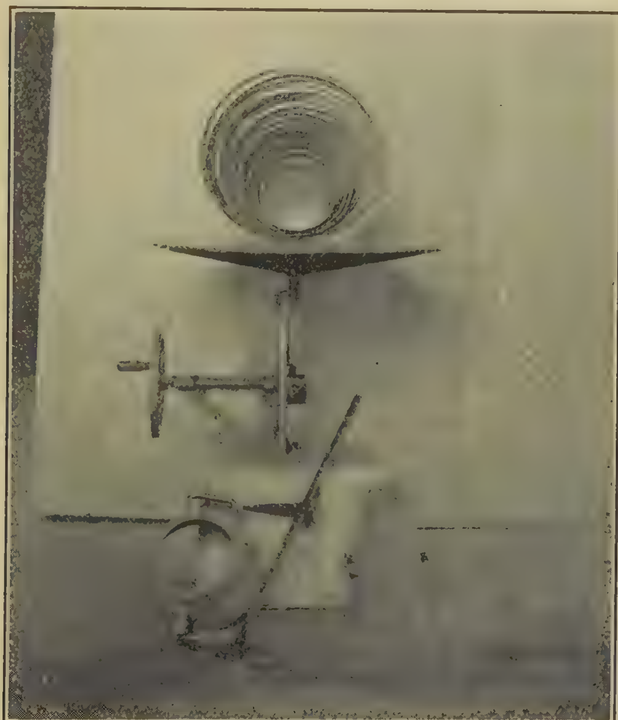

FIG. 2.—FLOW TABLE AND CUTTERS.

to use a concave top table 9/16 in. deep at the center and an inverted cone top table 2 in. low in the center. The results obtained with these irregular tops were no better than those obtained with the flat top table and are omitted from this report.

Using the flat top table the study of the distribution of the ingredients of the concrete was made as follows: The batch was mixed and the flow test made in the usual manner except for the change in height of drop, that is, the concrete was placed in the standard mold, the mold removed and the table caused to drop 15 times in about ten seconds by means of a crank and cam. If the maximum and minimum diameters showed a differ-

ence greater than ¾ in., the test was made on a new batch. The center of the mass was located by striking arcs from the edge. From the center four points were located in the mass which determined the position of the cutter which would divide the area, covered by concrete, into two equal parts. The inner portion was a circular area. The outer was a circular ring. The volume and weight of the concrete and the weight of aggregate

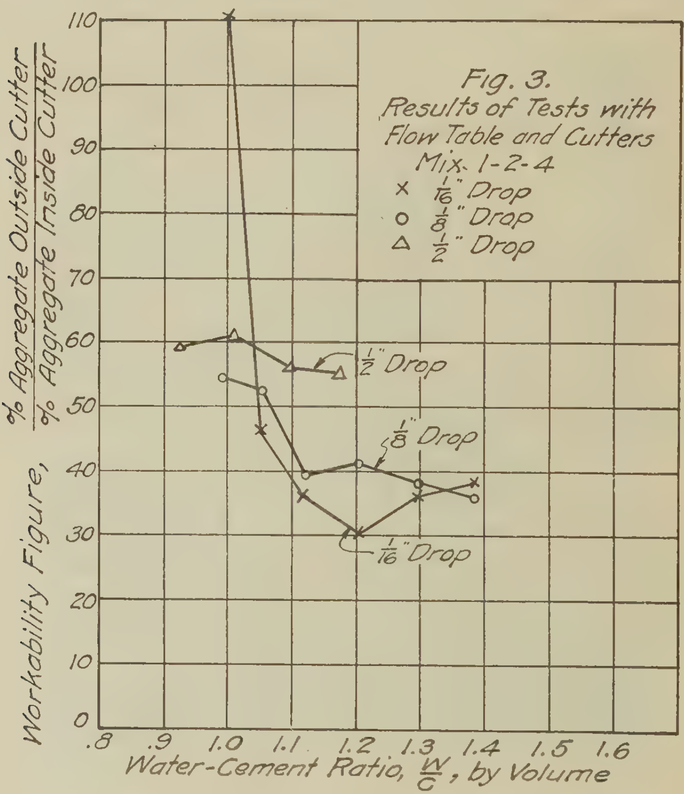

FIG. 3.—RESULTS OBTAINED IN FLOW TABLE AND CUTTER TESTS.

coarser than the No. 4 sieve in each portion were measured. A segregation figure was calculated as shown:

$$S = \frac{A}{B}$$

Where S = segregation figure

A = Per cent of the coarse aggregate in the specimen in the outer portion

B = Per cent of the concrete in the specimen in the outer portion.

This method did not show that there was segregation in any of the concretes, that is, S was equal, within experimental error, to unity.

A workability figure was calculated as shown:

$$W = \frac{C}{D}$$

Where W = workability
C = Volume coarse aggregate in outer portion
D = Volume coarse aggregate in inner portion.

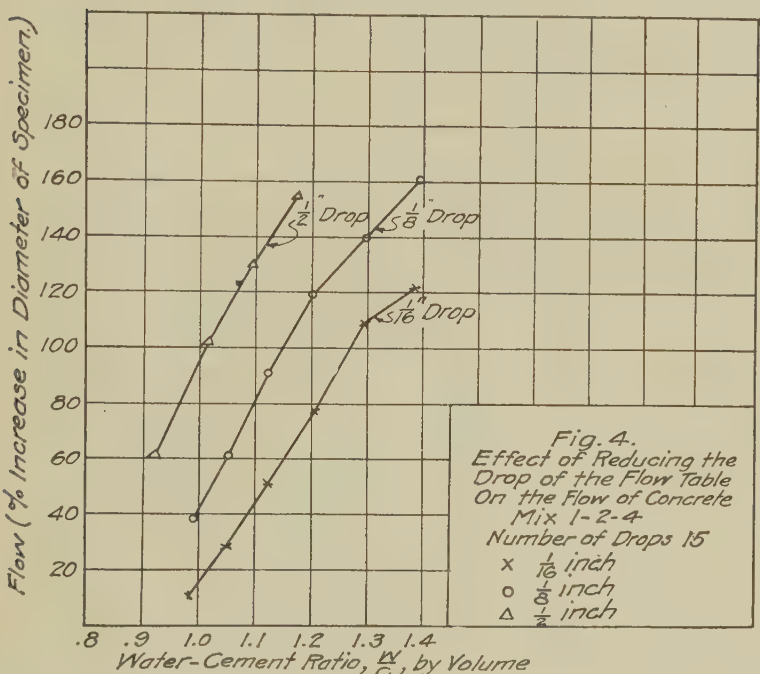

FIG. 4.—EFFECT OF THE REDUCTION IN THE DROP OF THE FLOW TABLE ON THE "FLOW" OF THE CONCRETE.

The results obtained are shown plotted in Fig. 3. Other workability figures were calculated by a similar relation but using instead of volumes of aggregates, volumes of concrete in one case and weights of concrete in another case. These figures were similar but a little lower in value than those shown. A study of these results (Fig. 3) shows that using the ½ and ⅛-in. drops on the flow table this method did not measure the change in the concrete due to a change in the mixing water. The results obtained using a 1/16-in. drop on the flow table give a curve of the type expected, that is, a moderately dry or excessively wet concrete is less workable than

a moderately wet concrete. The few tests made with concrete of different proportions indicated that this method would show very little difference in the workability of concretes when large differences caused by other changes than the water content are apparent from the ease of mixing and placing. Because of the difficulties we had in making these tests it is also believed that the personal element enters into this method of using the flow table for measuring workability, even with the 1/16-in. drop, to such an extent that it is unsatisfactory as a standard test.

Incidental to these measurements the flow of the concrete was measured. The values for the flat top table with the ½, ⅛ and 1/16-in. drop are shown plotted in Fig. 4. These results show the importance of having a

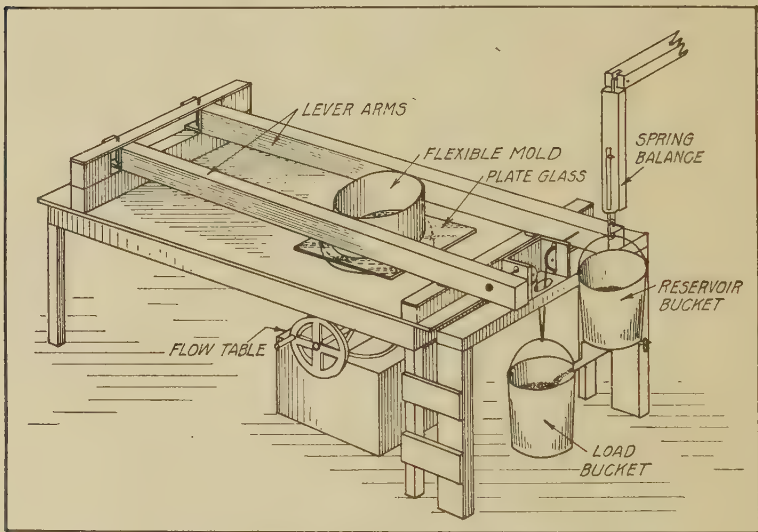

Fig. 5 – The Deforming Cylinder Apparatus.

uniform and accurate drop at all times, when measuring consistency on the flow table. A small deviation from the height of drop causes a marked change in the results.

Study of a New Method of Measuring Workability: The Deforming Cylinder Apparatus.—Dr. L. B. Tuckermann, of the Bureau of Standards, suggested a new method of applying the principle of mass deformation which was originally used by Prof. A. N. Talbot (Ref. 26) in his early work in this field. He suggested that the resistance of a mass of wet concrete to deformation under a laterally applied load might be an indication of its workability. A crude apparatus was built to try out the idea and considerable time and study has been given to the development of this apparatus. In all, three methods of filling the cylinder, four methods of

applying the load, four sizes of cylinders and four methods of reducing the friction between the specimen and the base plate have been tried.

The apparatus as now constructed is shown in Fig. 5. It consists of a flexible metal mold, of No. 30 gage phosphor bronze, 10 in. in diameter

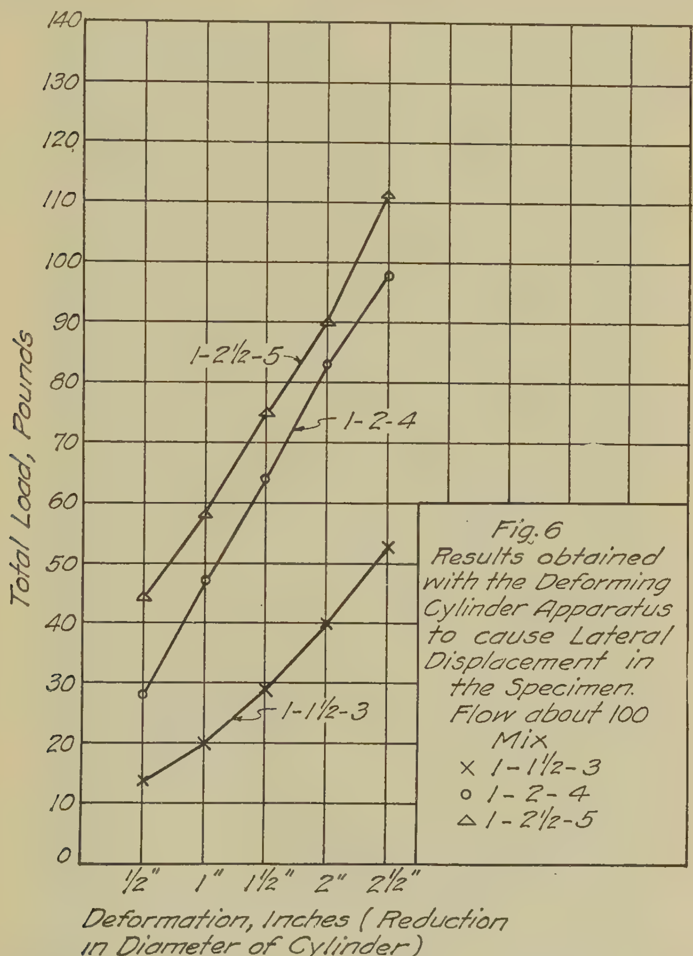

FIG. 6.—RESULTS OBTAINED WITH THE DEFORMING CYLINDER APPARATUS TO CAUSE A LATERAL DISPLACEMENT IN THE SPECIMEN.

and 6 in. high. The bottom edge is fitted with a rubber rim to reduce the friction when the cylinder is deformed and moved across the plate glass base. The upper edge is fitted with two small lugs to facilitate the measurement of the deformation. The base plate is a section of plate glass

12 in. wide, 14 in. long and ¼ in. thick. It is placed on top of a flow table with a top 12 in. in diameter. The deformation is caused by the application of the load to the lever arms on each side of the cylinder mold at one-third the height of the specimen. The lever arms are 2 x 4 in. in

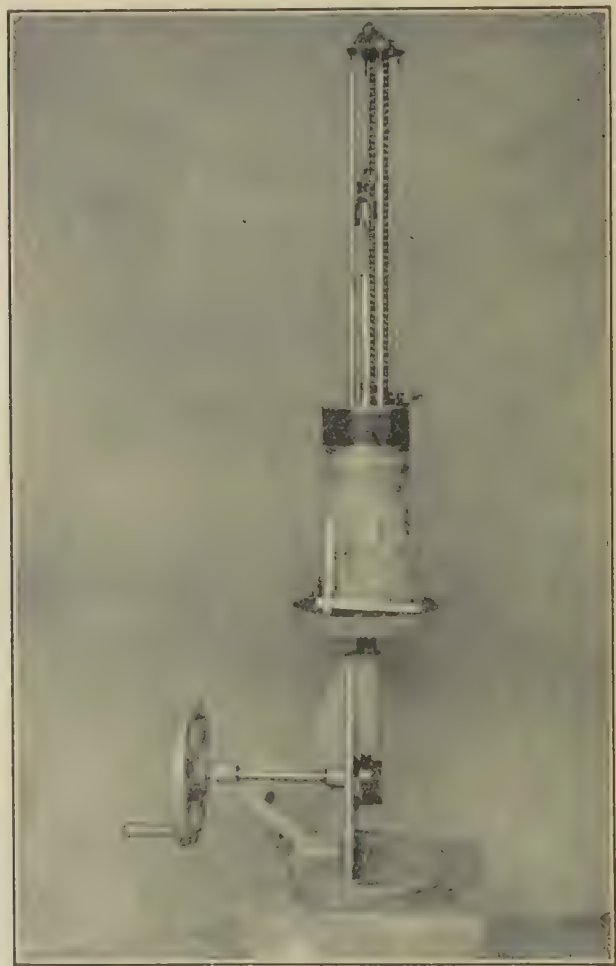

FIG. 7.—THE IMPROVED SINGLE ROD PENETRATION APPARATUS.

cross-section and 5 ft. long, pivoted 13½ in. apart and attached at one end of the table supporting the apparatus. The load ends of the levers are attached to the load bucket by means of a cord over pulleys in such a manner that both levers are forced to move the same distance when the load

is applied. The load is applied by allowing shot to run into the load bucket from the reservoir bucket suspended on a spring balance. The reservoir bucket is fitted with a valve in the bottom similar to the valve in a briquet testing machine. The load is measured by the difference between the weight of the shot in the reservoir bucket at the start of the test and the weight of shot remaining at the time a reading is taken.

The test is made as follows: The concrete is mixed in the usual manner and placed to a depth of slightly more than 5 in. in the flexible mold. It is jolted by means of the flow table, with a 1/10-in. drop, 30 times to settle the concrete to a height of approximately 5 in. and to overcome any personal errors or other inaccuracies in placing. A few trials enabled us to estimate very closely the quantity of concrete needed to fill the cylinder. The lever arms are then placed against the sides of the mold and the load

TABLE 1.—RESULTS OF TESTS WITH THE DEFORMING CYLINDER APPARATUS

Batch No.	Mix by Volume	W/C	Flow, per cent	Pounds Load Required to Cause Inches Deformation, etc.				
				½ In.	1 In.	1½ In.	2 In.	2½ In.
1	1:1½:3	0.911	103	17	..	33	40	56
5	1:1½:3	0.911	102	12	17	23	38	42
9	1:1½:3	0.911	105	13	23	31	42	61
Average	1:1½:3	0.911	103	14	20	29	40	53
2	1:2:4	1.13	106	29	51	70	86	86
6	1:2:4	1.13	107	29	49	65	84	104
10	1:2:4	1.13	109	25	41	57	78	105
Average	1:2:4	1.13	107	28	47	64	83	98
4	1:2½:5	1.37	99	49	61	78	92	110
8	1:2½:5	1.37	96	49	63	75	84	106
12	1:2½:5	1.37	92	34	50	71	93	117
Average	1:2½:5	1.37	96	44	58	75	90	111

is applied by starting the flow of shot. The load required for each ½-in. increment reduction in diameter of mold is read.

The average results of the last series of tests made with this apparatus are shown plotted in Fig. 6. The individual reading from which these curves are plotted are given in Table 1. The difference in workability, as measured by this apparatus, between the 1 : 2 : 4 and 1 : 2½ : 5 mixes is so small as to throw doubt on the value of the apparatus used. There was a wide variation between the individual tests on the same batch as may be seen from Table 1. The load required to reduce the diameter of the cylinder 1½ in. appeared to give the best value for a workability figure. Other tests made with this apparatus showed that it was not sensitive enough to show any effect of a longer time of mixing on the workability. However, it is believed that an apparatus using the same principle as this one might be developed into a satisfactory device for measuring workability.

The Improved Penetration Apparatus.—After completing several thousand tests, on all other methods suggested for measuring workability, we again took up the study of the Pearson and Hitchcock penetration appara-

tus (Ref. 8, 9) with a view of improving it. This apparatus consisted essentially of a metal rod ¾ in. in diameter, 20 in. long, held by means of a spider and sleeve in a vertical position over the center of a 6 x 12 in. cylinder mold which was mounted on a small flow table. The concrete to

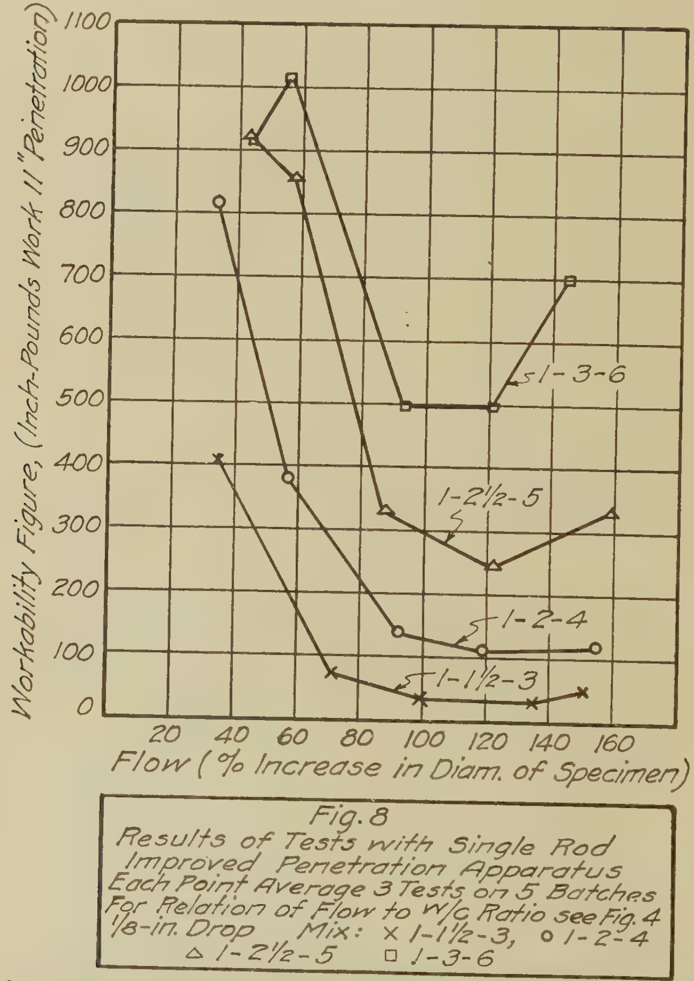

FIG. 8.—RESULTS OF TESTS MADE WITH SINGLE ROD IMPROVED PENETRATION APPARATUS.

be tested was placed uniformly in the mold and compacted by 30 drops of the flow table, then the rod was inserted in the sleeve and lowered into the concrete until it came to rest under its own weight. The mold was then jarred by raising and dropping the flow table. The number of drops re-

quired to cause the rod to penetrate 10 in. into the concrete was a rough index of the workability.

The continual jarring, in this method of making the test, caused a non-uniform segregation and compacting of the aggregates in the concrete being tested. This action produced extreme variation in the results and had a marked effect upon the precision of the test. It was thought, that if an external force could be used to drive the rod into the concrete, the apparatus could be made to function satisfactorily, the precision of the results would be improved, and the workability figure could be expressed in actual inch pounds of work.

A miniature pile driver was first constructed to drive the rod into the concrete. The tripping device was set in a fixed position so that the hammer fell through varying distances in driving the rod into the concrete. This variation in the height of fall of the hammer made it difficult to calculate the energy expended which was taken as a workability figure. In other respects this was a great improvement over the original apparatus.

An improvement, Fig. 7, was made to this apparatus by attaching an automatic tripping device to the hammer so that the height of the drop of the hammer was kept constant. The modification which made this possible was suggested by Dr. L. B. Tuckerman of the Bureau. With this apparatus the energy absorbed by the concrete could be easily calculated by the equation:

$$W = p(R_w + H_w) + nhH_w$$

Where p = depth of penetration in inches
R_w = weight of rod in pounds
H_w = weight of hammer in pounds
n = number of blows struck
h = height of fall in inches

A number of tests were made with the apparatus thus modified on concretes varying in richness from 1: 3: 6 to 1: 1½: 3. The results of these tests are shown in Fig. 8. A study of these results show that the energy so calculated on the modified apparatus measured either workability or a parallel property of the concrete for the results line up in the same order as is observed in the field. For this reason we shall call this energy the "workability figure" for the concrete. The different concrete mixes have different workability figures when mixed to the same consistency as measured on the flow table. A 1: 1½: 3 concrete is easier to place than a 1: 2: 4 concrete of the same flow and the rich mix has a lower workability figure than the lean mix. Similar comparisons can be made with the other mixes. A dry concrete (flow 40 to 60) or an excessively wet concrete (flow 140 to 160) is more difficult to place and has a higher workability figure than a moderately wet concrete (flow 100 to 120). The use of an excessive quantity of mixing water has a greater detrimental effect on the lean mixes than on the rich mixes. (Apparently it causes a more complete segregation of the aggregates in the lean mixes.)

A study of the individual tests with the single rod apparatus showed a rather wide variation in the results. Occasionally when testing the very rich and workable mixes the rod would penetrate more than 11 in. under its own weight. When this occurred it was impossible to obtain a workability figure. In order to reduce the variation of the individual results and to obtain a workability figure for all mixes a three-rod plunger was substituted for the single rod. The three rods were each ½ in. in diameter and rigidly fastened together at the top so that they could be driven down as a unit. A series of tests were made with the three-rod and the single-rod apparatus with the concrete specimen in a 6 x 12-in. mold. The results are shown in Fig. 9.

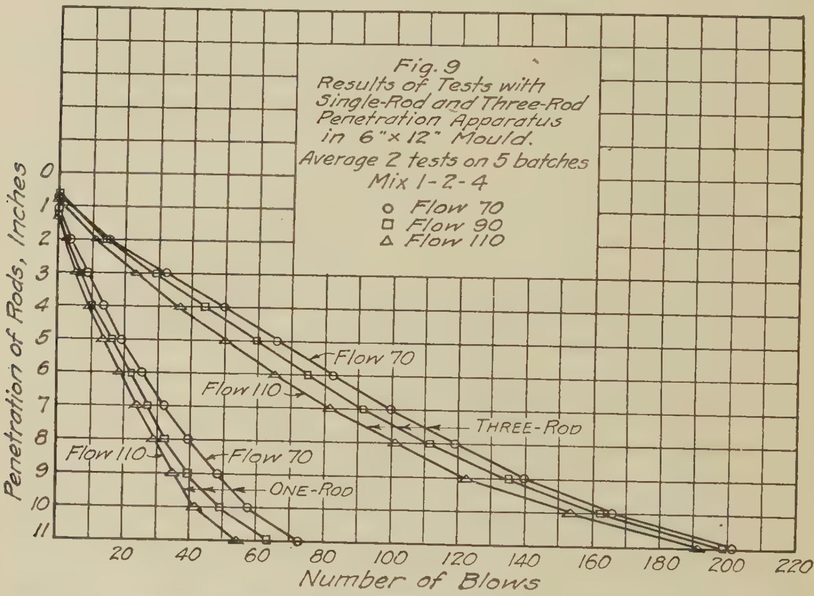

FIG. 9.—RESULTS OF TESTS WITH SINGLE ROD AND THREE-ROD PENETRATION APPARATUS IN 6 X 12-IN. MOLD.

A study of these results showed that the three-rod plunger did not measure as great a difference between the same concrete with a flow of 70, 90 and 110 as was measured by the single-rod apparatus. The difference when the rods had penetrated about 7 in. (see Fig. 9) was greater but at the depth of 11 in. the three-rod plunger showed very little difference between the three concretes. From observations made during the test it was concluded that the space between the walls of the cylinder and the penetration rod had a very marked influence on these results. Accordingly a cylinder 8½ x 12 in. was built and the driving mechanism adapted to it. The program of tests with the 6 x 12-in. mold was duplicated with the

8½ x 12-in. mold. The average curves obtained from the results obtained in these tests are shown plotted in Fig. 10. To avoid confusion in the curves of Fig. 10 are drawn with different abscissas instead of the same as in Fig. 9. In these tests there was a greater difference in the workability figures obtained with the three-rod apparatus than with the single-rod apparatus. These figures were, however, of the same relative values and indicate that the three-rod apparatus is more sensitive than the single-rod apparatus. The curves of Fig. 10 continue to diverge at the 11-in. penetration as contrasted with the curve of Fig. 9 showing that the effect of the walls of the cylinder on the test has been corrected.

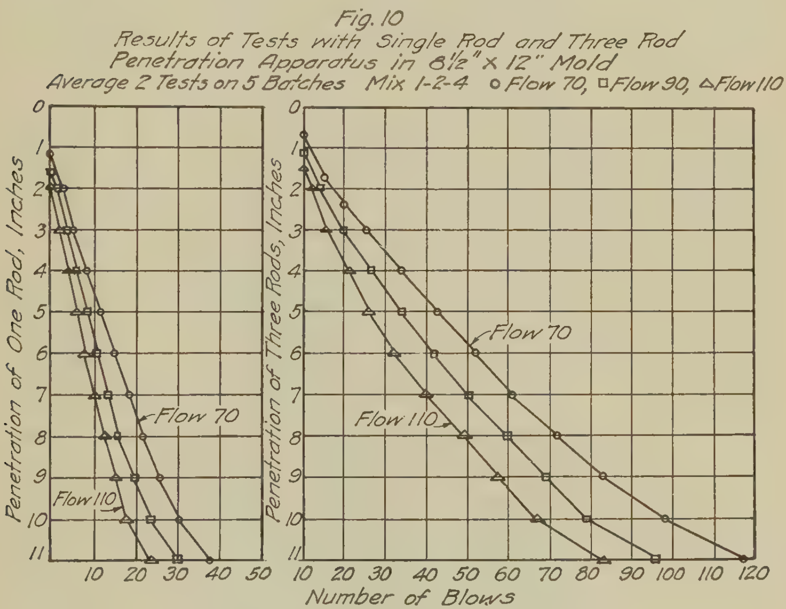

FIG. 10.—RESULTS OF TESTS WITH SINGLE ROD AND THREE-ROD PENETRATION APPARATUS IN 8½ X 12-IN. MOLD.

The precision of the results obtained with the large mold and the three rods does not appear to be particularly better than with the one rod. Complete data are not yet available but it is believed that an improvement has been accomplished. In the first place the three-rod plunger will not penetrate 11 in. into the concrete specimen under its own weight. The range of applicability of the test seems to have increased, that is, it is believed that a wider range of gradation of the aggregates can be tested. The larger mold requires a larger quantity of concrete and it appears to be more easily placed to form a uniform specimen. The three-rod apparatus appears to be more sensitive than the single rod because the difference in

the workability figures is larger but of the same relative order as those obtained with the single-rod apparatus.

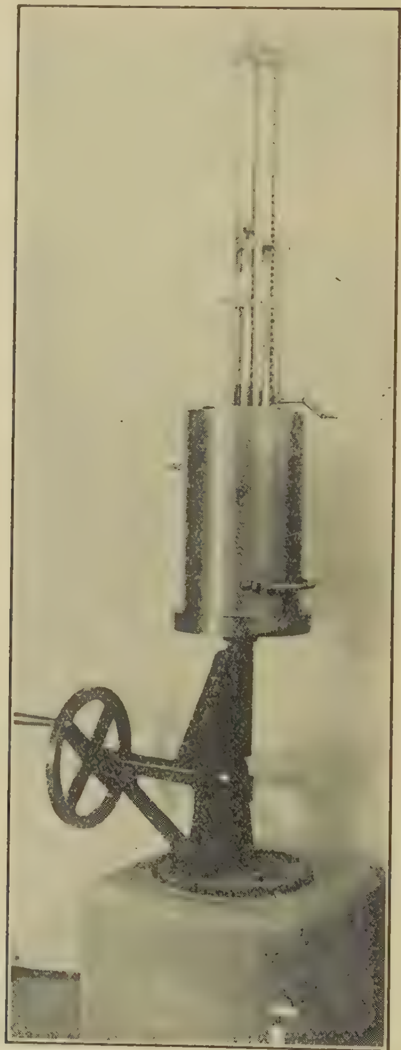

FIG. 11.—ENLARGED AND IMPROVED PENETRATION APPARATUS.

In order to compare the results of tests made with the improved apparatus, Fig. 11, with those obtained by Pearson and Hitchcock tests

were made using hydrated lime, Florida kaolin, celite and additional cement to improve the workability. The mix used was 1: 2: 4 (by volume). The cement had a fineness of 85.9 per cent and passed all the requirements of the Bureau of Standards for portland cement, Circular 33, fourth edition. The fine aggregate was washed river sand having a fineness modulus of 2.75 and the coarse aggregate was gravel graded from No. 4 to ¾ in.

A test with the enlarged penetration apparatus was made as follows: A batch of material which, when mixed, would a little more than fill the mold, was carefully prepared. This was mixed by hand in the usual manner for two minutes from the time the water was added. The flow was then measured and the concrete again slightly mixed to insure uniform distribution of the materials. The penetration mold was filled at once, to within ¼ in. of the top and jolted 30 times to settle the concrete. The driving mechanism (which was supported from a pulley directly over the mold) was lowered into place, the penetrating rods being raised at the same time so as to be free from the concrete. With the hammer resting on the spider holding the rods, the rods were lowered to the zero mark and released. The rods were then driven down by means of the hammer and the number of blows required for an 11-in. penetration, recorded. The mold was emptied, the concrete remixed slightly and the test repeated. The workability figure was taken as the work done by the rods and the hammer in accomplishing the penetration. (See equation given above.) The tests compared were made at the same time and consisted of three penetration tests on each of five batches of concrete for each condition. Results of these tests are given in Table 2 and are shown plotted in Fig. 12.

TABLE 2.—RESULTS OF TESTS WITH THREE ROD PENETRATION APPARATUS

Kind of Admixture	Per Cent by Weight of Cement	W/C	Consistency		Workability Figure
			Flow, per cent	Slump, inches	
None	....	1.005	90	5.8	395
Kaolin	3.19*	1.004	86	5.7	316
Kaolin	6.38	1.110	87	6.5	186
Hydrated Lime	3.19*	1.027	87	6.1	357
Hydrated Lime	6.38	1.059	88	6.6	291
Hydrated Lime	9.58	1.081	92	7.0	233
Celite	3.19*	1.119	89	6.6	223
Cement	25.00	0.864**	92	6.8	236

* 3 lbs. per bag of cement; ** Based on all the cement used.

Pearson and Hitchcock found (Ref. 9) that the workability of a concrete mixture was about equally improved by the addition of 9 lb. of hydrated lime, 6 lb. of kaolin or 3 lb. of celite per bag of cement or by the use of 25 lb. of additional cement. A study of the results of the tests in Table 1 and Fig. 12 showed that, as measured by the three-rod penetration apparatus, the workability of a concrete mixture was about equally improved by the addition of these fine materials in proportions used by

Pearson and Hitchcock. In the cases where smaller amounts of the admixtures were used the results seem to be more consistent than those obtained by Pearson and Hitchcock.

Résumé.—The apparatus developed in the study of the plastic properties of neat cement pastes could not be adapted to measure workability of concrete.

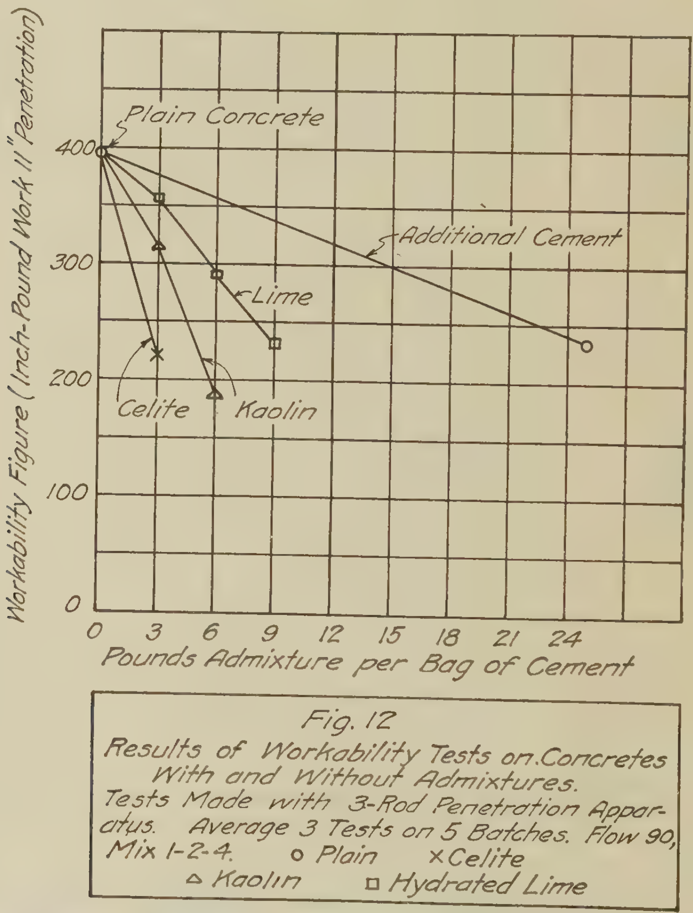

FIG. 12.—RESULTS OF TESTS FOR WORKABILITY OF CONCRETES WITH AND WITHOUT ADMIXTURES.

The tests made with the modified flow table and cutters did not develop a satisfactory method for measuring workability. The results of individual tests varied over a wide range and the difference in workability between mixes of different proportions was small. The personal element

enters to a large degree into this method of making this test for workability.

The results of tests made with the deforming cylinder apparatus were somewhat discordant among themselves. The differences in workability between the lean mixes of different proportions as measured by this apparatus were not large enough to be distinct and conclusive. The test was not sensitive enough to show the effect of an increase in mixing time on the workability of the concrete. However, it is believed that this method might be developed into another satisfactory method for measuring workability.

The tests with the penetration apparatus have brought about the modification and improvement of this apparatus for determining an index of the workability of a concrete mixture. The use of the pile driver mechanism has eliminated the compacting of the aggregates which often prevented the rod from penetrating to the desired depth and which also decreased the precision of the test. The rods can now be driven to the desired penetration. The use of the smaller weight of rods requires a sufficiently large number of blows to give a workability figure for the very workable concrete mixtures. The larger mold has materially expedited the placing of the sample and lends itself to a more satisfactory distribution of the materials in the specimen thus reducing the personal element in the test.

In order that a satisfactory workability figure may be obtained, it is believed that the average of three tests on each of five similar batches is necessary. Subsequent tests have shown that average workability figures obtained with this number of tests check very closely when conditions and manner of making the test are carefully controlled.

Compared with variations in test methods used to measure the other properties of concrete specimens, the variations in this method of obtaining a workability figure are not excessive.

The materials employed, thus far in developing this apparatus, have been limited to the sizes of aggregates and mixtures commonly used in reinforced concrete. The future work should be extended to the use of larger aggregates. The apparatus can probably be further improved in precision by increasing the size of the test cylinder and further work along this line is planned.

Each step in the development and improvement of the penetration apparatus has increased its reliability as a device for determining an index of the workability of a concrete mixture. It is believed that the present form gives the most satisfactory index of workability yet developed. It is suggested that it be studied further as a standard for evaluating the workability of concrete.

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Cement as a Factor in the Workability of Concrete.[1]

By P. H. Bates[2] and J. R. Dwyer.[2]

In this paper will be discussed some of the data which have been obtained at the Bureau of Standards in its study of the workability of cement pastes. So far the studies have not been extended to concrete mixtures. However, if it can be demonstrated that there are differences in the workability of pastes, then it would follow that these different workabilities must also affect this property of concrete. Since it has not been demonstrated to any degree of satisfaction that pastes made from different cements do have different degrees of workability it was thought desirable to attempt to measure this property quantitatively and then at a later time to demonstrate the effect of the pastes made from different cements on the same dry mixture of aggregates.

We have had considerable thought given to the effect on workability by the character of the coarse aggregate,—its size, shape and the amount compared with that of the fine aggregate, the relative amounts of the total aggregates and cement, the amount of water, the degree of mixing, etc. On the other hand, we have had little thought given to the part that cement may play in inducing workability in concrete. It may have been assumed that since there is a certain standard for cement to which all portland cements produced will conform at practically all times these same cements will be alike in all other properties although these other properties may not be indicated in this standard. This may be the reason why there has been little discussion of the influence of cement on the workability of concrete. But cement as a factor in this desired property of concrete should also be a matter of primary importance because it is the factor which confers to the water used in mixing the concrete those qualities which are essential for making workable mixtures.

The dry coarse aggregates, regardless of whether rounded or angular in shape, do not have those characteristics which are associated with workability. When well graded so as to produce maximum density they do not flow as readily as they do when they are sized within narrow dimensional limits, but under these latter conditions they lack density to too great a degree to be acceptable as aggregates. When composed of different sized particles, as they usually are, they are also subject to ready segregation of the coarse particles on handling, which in turn requires more working to obtain the needed uniformity. When mixed with fine aggregate to secure

[1] Publication Approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.
[2] Bureau of Standards.

greater density, increased likelihood of segregation again results, hence decreasing the desired workability. If water is added to the mixed aggregates in any marked amount the likelihood of segregation is further increased, although there may now be obtained the quality of more ready flowability. It is only when the cement is used in connection with the water and the mixture of aggregates that we begin to obtain workable concrete in its true sense—the somewhat sticky, relatively easily deformable and readily flowable non-segregating mass of water, cement and aggregates.

Experimental procedure to measure workability of cement pastes has not been developed to any great degree. Considerable work has been done on plastering mortars in this respect, particularly by Emley,[1] whose last design of apparatus has been accepted by the American Society for Testing Materials as the means of determining the plasticity of hydrated lime and is so indicated in its standard specification C6-24, "Standard Specification for Hydrated Lime for Structural Purposes." This same equipment has been used by Weymouth in determining the plasticity of brick mortars.[2] Emley, in his paper above referred to, indicates the variety of apparatus which can be used for the purpose in question and discusses the limitations of each, but he was dealing with materials which can be considered as being quite plastic and very fine grained as compared with portland cement pastes. It would seem that lime pastes and portland cement pastes are distinctly different and it would possibly be better to refer to the latter as being more or less workable rather than plastic; hence it was not thought advisable to use in the present study his plasticimeter or many of the other types of apparatus referred to in Emley's paper, although they may be studied in later work. In fact, in the present paper it is rather the intention to show that different cements have widely different working qualities, and to show this with equipment which is at hand in most laboratories or can be readily constructed therein. Three methods were used in this work.

Extrusion Apparatus.—The first method was carried out with the apparatus shown in Fig. 1. This is an adaptation of a device used in a number of tests in clay working research. Reference to it appears as far back as 1914.[3] The particular apparatus used in the tests herein described was designed by Mr. Grunwell, of the Bureau staff, in connection with some studies of the extrusion of clays. Its adaptation to the study of portland cement paste required considerable experimentation before suitable size of orifice and speed of piston were selected. As a result of this study, the tests herein reported were made using a ¾-in. round orifice and a piston speed of ¾ in. per min. Two consistencies only were used, 25 and 30 per cent of water, by weight of cement. The results obtained using 25 per cent water only appear in Fig. 2.

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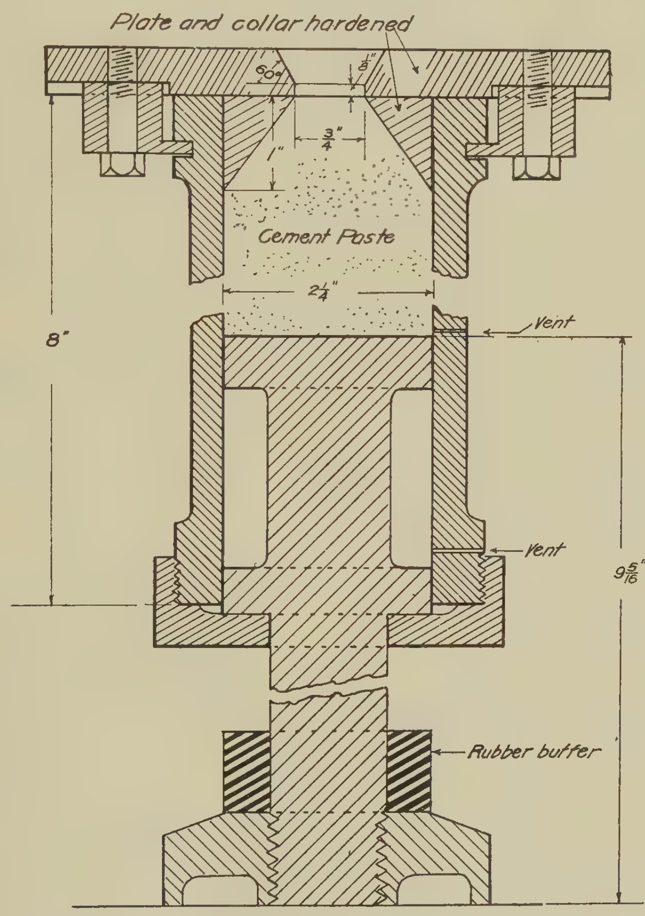
[1] "Measurement of Plasticity of Mortars and Plasters," Emley, Technologic Paper No. 169, Bureau of Standards.

[2] "Improved Brick Mortars," Weymouth, *Proceedings*, American Society for Testing Materials, Volume 27, 1927.

[3] "Flow of Clay under Pressure," Bleininger and Ross, *Transactions*, Amer. Cer. Soc., 1914, page 392.

The method of procedure was as follows: Pass cement through a No. 20 sieve, pour water at 20 deg. C. into the cement, allow ½ min. for absorption, mix by hand for 1 min., fill cylinder, place between the heads



Extrusion cylinder as used in studying the flow of neat cement pastes through orifices when under pressure.
Fig. 1.

of a low capacity compression testing machine, and apply pressure at 3½ to 4 min. after addition of water. The weighing beam was kept balanced and readings taken ordinarily at 1/10 in. intervals until the piston had

traveled 3 in., or the capacity of the machine, 2,000 lb., had been reached.

In Table 1 are given some few characteristics of the cements in question which might influence their behavior in the extrusion cylinder. There seems to be no constant relation between the extrusion test results and either the fineness on the 200 sieve or the setting time. In general, it appears that the load at a given point increases with the water requirements of the neat cement in so far as the latter is measured by the Vicat test for normal consistency. However, there are some wide exceptions to this statement. Cements 3, 4, 1 and 6 require approximately the same percentage of water for normal consistency. Cement 9 requires about 24

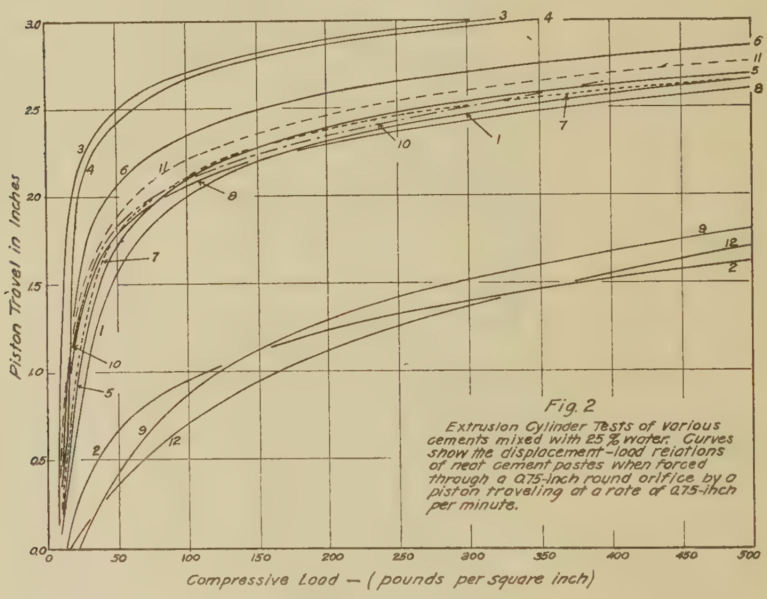


Fig. 2

Extrusion Cylinder Tests of various cements mixed with 25 % water. Curves show the displacement–load relations of neat cement pastes when forced through a 0.75-inch round orifice by a piston traveling at a rate of 0.75-inch per minute.

per cent; cement 2, 24½ per cent; and cement 12, 27.6 per cent. However, these three latter cements give curves which are very closely related as to position. Relatively cements 9 and 12 would be considered highly plastic, while cement 8 would be considered as of the opposite nature. Their positions in Fig. 2 should be noted.

Apparently the ability of a cement to hold the mixing water is indicated by these results. This is also made very apparent by reference to the portions of the specimens removed from the cylinder at the end of the test, as shown in Fig. 3. The specimens in the upper row had contained 25 per cent water; those in the lower row, 30 per cent water. At the end of the test the larger portions of the specimens were very hard and dry. The ability of a paste or mortar to hold water, especially when in contact

with an absorbent base, as a brick or a coat of plaster, is of extreme importance in a study of workability of these materials. Such are generally placed upon absorbent bases, and to be truly workable to the artisan they must not readily give up water to the base but must remain plastic for a period sufficiently long to permit of their proper usage. This fact was borne in mind by Emley in developing his plasticimeter and the measurements with his apparatus are made on the paste after placing on a standard absorptive base. One of the valuable features of the extrusion cylinder seems to be its ability to demonstrate the marked difference in the pastes in their remaining fluid during the test in question. As indicated by Fig. 3, some pastes remain readily flowable and are largely forced out of the cylinder before the capacity of the machine is reached. Others seem to act as a filter cake and allow a small portion of the paste and a large percentage of the water from the residue to pass out of the orifice,

TABLE 1—FINENESS, NORMAL CONSISTENCY, AND TIME OF SET OF THE CEMENTS USED.

Cement number	Fineness	Setting time Initial Hrs. Min.	Setting time Final Hrs. Min.	Normal consistency
1	82.6	3–15	6–00	22.5
2	92.7	6–00	7–45	24.4
3	81.7	3–45	6–00	22.7
4	80.8	3–00	5–00	22.6
5	82.6	2–30	5–00	22.3
6	87.3	4–15	6–45	22.4
7	84.0	4–15	7–15	23.0
8	79.7	5–15	6–45	23.4
9	90.8	4–00	5–45	24.1
10	84.4	4–45	6–30	23.1
11	88.6	3–15	5–15	22.8
12	99.3 (a)	3–15	6–15	27.6

(a) Passing No. 325 sieve = 97.8%.

leaving a large part of the original charge in the cylinder as a dry compact mass. In passing, it might be stated that cement 10 is used in commerce for some special work where its water-holding properties are important.

Ball Plasticimeter.—The ball plasticimeter as used in this study is shown in Fig. 4. This is not the first attempt to apply it to a study of neat cement. The apparatus was largely built by Messrs. Conahey and Smith for work at the Bureau in connection with their paper in this symposium. Essentially the test consists of determining rate of shear of the paste effected by various loads applied to pull the ball out of the paste.

The process as finally developed was as follows: The cement was passed through a No. 20 sieve and then gradually added to 36 per cent of water* at 20 deg. C. One-half min. was allowed for absorption. The paste was then thoroughly mixed with a trowel for 3 min., after which it was allowed to stand for 1½ min. Then the paste was remixed for 1 min.

* This amount of water was used because any large increase or decrease resulted in conditions unfavorable to the attaining of satisfactory working consistency; that is, the paste was either too dry or too wet.

and poured into the cylinder. The ball was then placed and centered by means of a rod, and a load, estimated as appropriate, was placed in the pan. The pan was then released, and the time of passing over a 30 cm interval shown in Fig. 4 was determined. The test was then repeated with smaller weights, endeavoring to secure about 5 different loads. The cylinder was not emptied and refilled as in earlier tests but before each placing of the ball the agitator was used, it being moved up and down through the height of the cylinder 5 times. Fifteen seconds were allowed each time between the placing of the ball and the release of the load.

When the first group of weights had been used, the process was repeated 5 times. Examination of the series then showed an increasing rate of shear for a given load, as the cement was subjected to reworking. This increase became less with repetition and the last 3 loadings usually

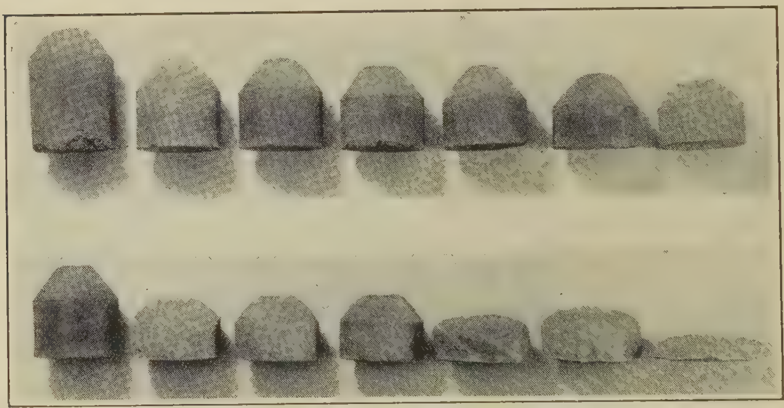


FIG. 3. PORTIONS OF SPECIMENS REMAINING IN CYLINDER AT COMPLETION OF TESTS.

Upper row prepared with 25 per cent water and lower row with 30 per cent water. Cement numbers from left to right are 2, 7, 5, 1, 6, 4, and 3.

showed comparatively close agreement in results. Of course, preceding the load at the end of a group the paste had more mixing or agitation than preceding the first load. The last 3 groups were arbitrarily taken as the basis for an average curve, due to a number of considerations. After these tests had been completed, the cement was then allowed to stand 30 min., when it was thoroughly agitated and series B, of three groups of the same loads as used at first, was carried out.

The curves resulting from these tests appear in Fig. 5 and give some comparison of the rate of shear at various loads under the conditions described. Cements 8 and 9 are extremes, but otherwise there is little in common between this method of investigation and the extrusion cylinder method, but there is a distinctly marked difference between the different

cements. Neglecting cement 9, producing the distinctly fatty or workable paste, we find that the load required to produce the same rate of shear with the different cements may vary as much as 250 per cent. The

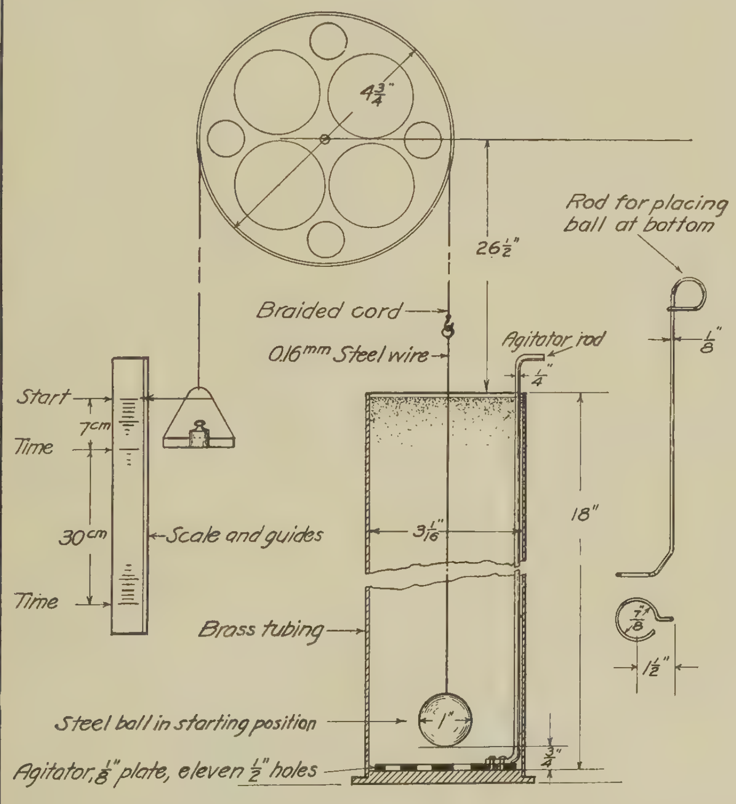


Fig. 4—Ball plasticimeter used in a study of cement pastes.

apparatus deserves further study, even though the results were not as precise as desired. It appears to be very sensitive, as indicated by the values obtained while the paste was aging. This however, should be considered an excellent quality in the apparatus and not a fault.

Capillary Tube.—The first attempts to use the flow of a cement paste through a capillary tube were not satisfactory, because of the residual coating on the inner surface of the burette which prevented accurate or ready reading of the position of the column, or of the amount of discharge. In order to eliminate this difficulty, the apparatus shown in Fig. 6 was devised. After some preliminary trials, it was decided to use a capillary having an internal diameter of 0.2 cm, and utilizing cement pastes of three consistencies, 39.5 per cent, 42 per cent and 44.5 per cent water.

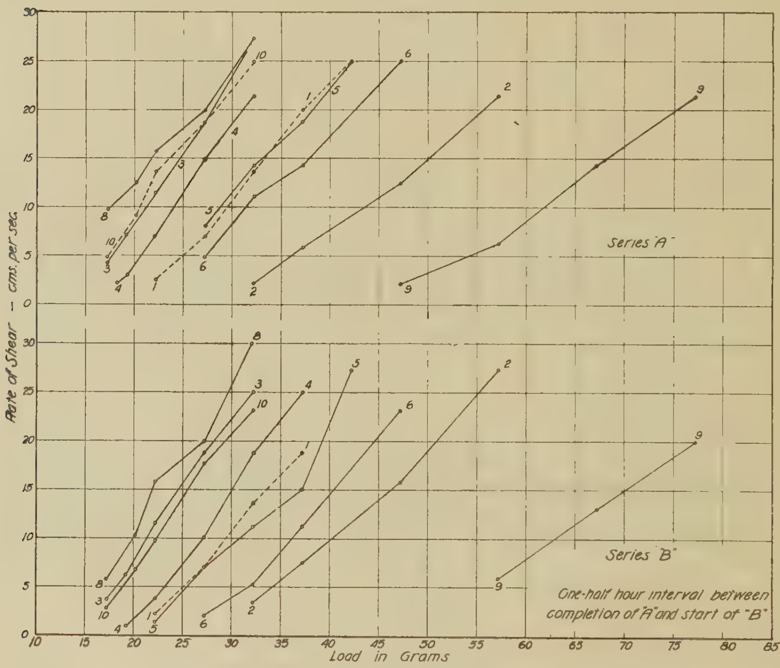


Fig. 5-Ball plasticimeter tests of neat cement pastes containing 36% water, by weight.

The process was as follows: Into the water at 20 deg. C. there was gradually poured the cement, which had passed a No. 20 sieve, the mixing being done by an electric stirrer, such as used by soda fountains. After 3 min. mixing, the paste was poured into the flask through the side opening. Kerosene was then poured into the top of this opening and the stopper was inserted. Then kerosene was poured into the top of burette until the latter was filled slightly above the top graduation. At 4 min. after mixing had been started, the cap was removed from the end of the capillary and the rate of flow noted, stop-watch readings being taken at various intervals.

The use of the apparatus as here designed involved some slight approximations in the calculations. The total effective pressure at the start was computed on the basis of 0.8 for relative density of the kerosene

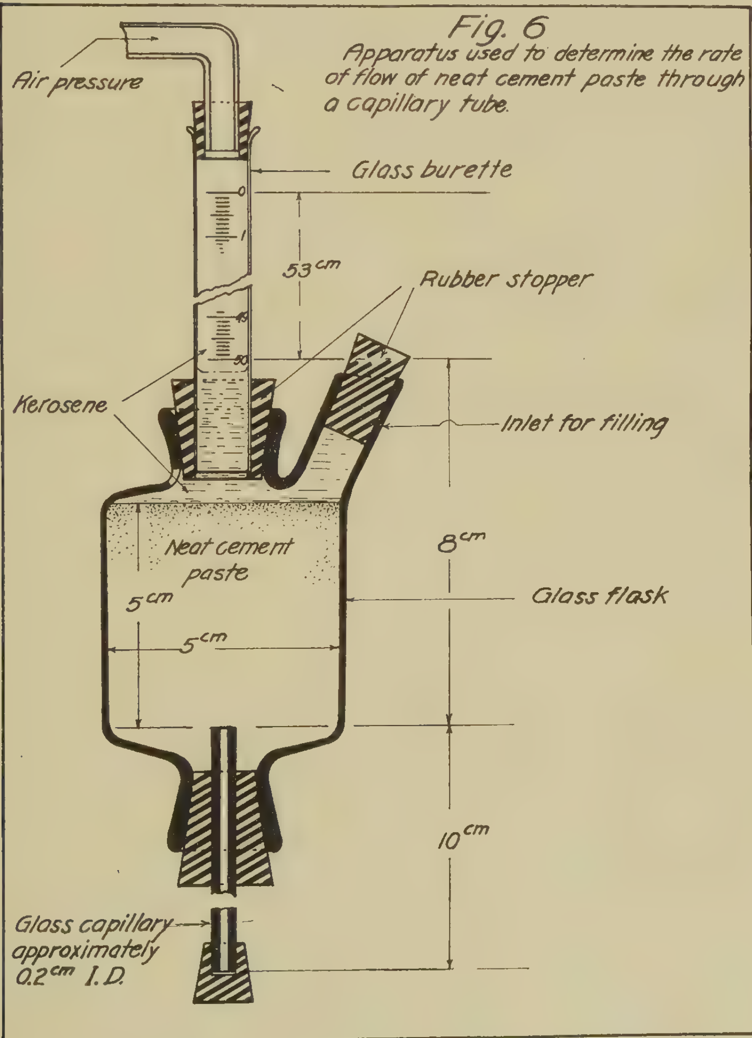


Fig. 6
Apparatus used to determine the rate of flow of neat cement paste through a capillary tube.

and 2.0 for the paste. To the pressures thus calculated was added the pressure of the air when any was used through the top connection. The changes in pressure were based only on the change in the height of the

kerosene column. The height of the cement paste above the end of the capillary changed during the test, involving some change in head due to the relative density of the paste and that of the kerosene. However, this change in height for the paste was not uniform for all cements and no correction was made for it. Some of the pastes on opening the capillary settled quite uniformly across the section of the flask while others developed conical or even narrow, deep craters, the paste adhering quite tenaciously to the glass sides.

Test series A was made with the pressure of the kerosene column only, and test series B with 1 lb. of air pressure added to the top of column. The results of the latter tests indicate that the use of a modified apparatus,

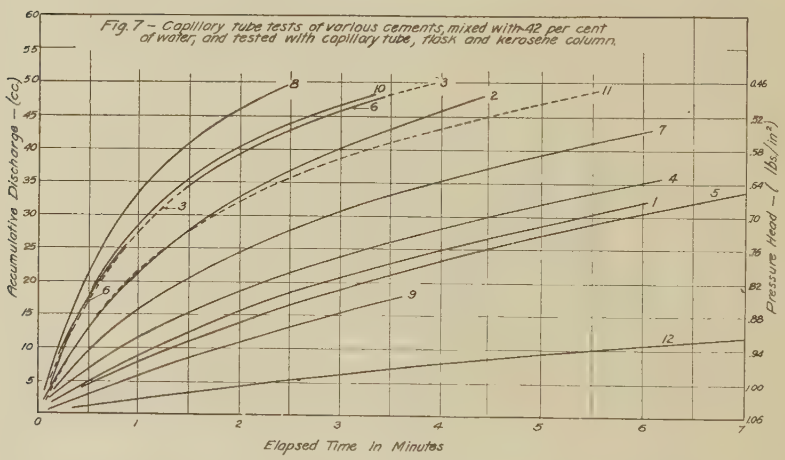


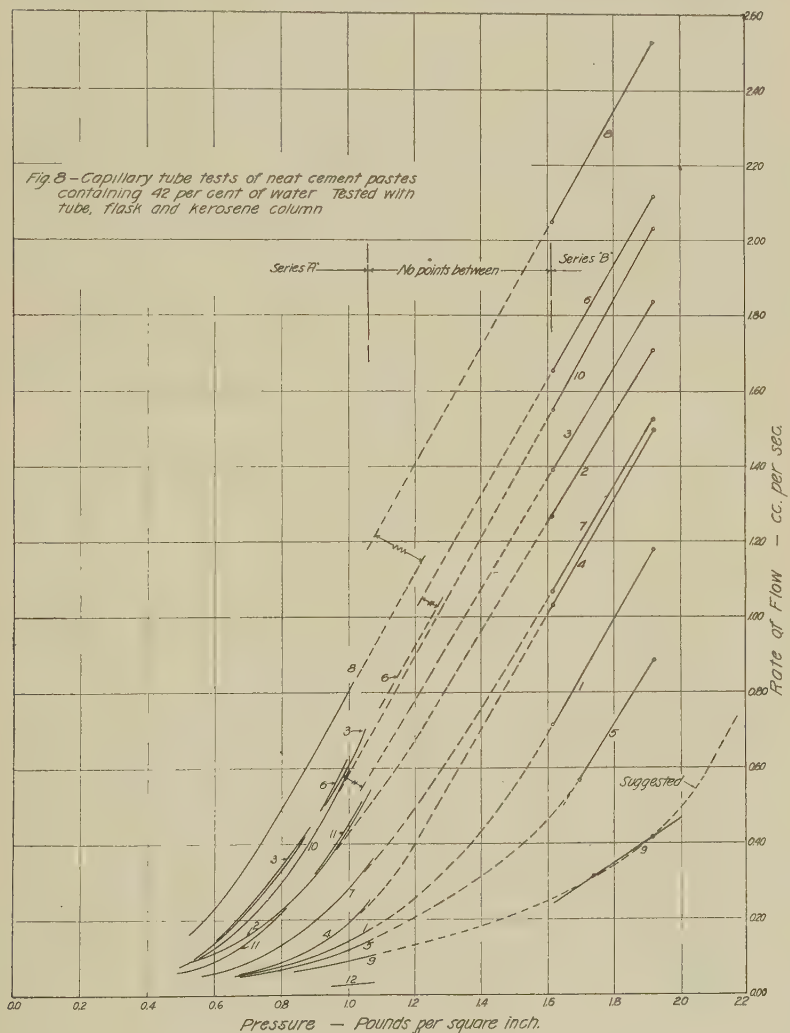
Fig. 7 – Capillary tube tests of various cements, mixed with 42 per cent of water, and tested with capillary tube, flask and kerosene column.

with considerably higher pressures, might eliminate some of the difficulties attendant on these tests, and would permit of determinations based on the time of flow of a considerable quantity of cement at any constant pressure head.

The results of the 42 per cent consistency tests are shown in Figs. 7 and 8. The data of series A were first plotted on the basis of the accumulative discharge at any time after the test had been started, and from the average curves of such plottings some of the calculations as to rate of flow were made. These accumulative discharge curves are shown in Fig. 7, and are of interest largely in that they show the relation between time and the quantity of paste discharged. When curves do not extend to the 50 cc discharge, it indicates that flow at that point was interrupted by the kerosene forcing its way through the paste, due to the uneven settlement of the latter.

The data from these tests were then examined from the basis of the rate of flow at various heads. The results of such study are shown in

Fig. 8. Note again the relation of cements 9 and 8, and bear in mind that these are produced by the same manufacturer but at two different mills. The former is considered by the manufacturer as being distinctly workable,



while the latter is considered decidedly the opposite. Note also that the rate of flow at a pressure of approximately 1.9 lb. per sq. in. ranged from 0.4 cc per sec. to 2.5 cc per sec. The parallelism of the curves for the

higher pressures used is quite striking, although its significance has not been particularly studied. The deviation from this characteristic for cements 9 and 12 has also not been thoroughly studied, but it suggests the use of higher pressures for all the pastes, and it would appear that with the higher pressures the curves for rate of flow from even the very unusually stiff and sticky pastes would parallel those having the more usual characteristics.

It was not expected that the three methods of investigation used would permit of arranging the cements in the same order. This was particularly true of the extrusion cylinder, which seems to measure very strikingly the water-holding properties of the paste more than the flowability. This is essential information in one phase of the workability studies and is apparently not indicated in the other methods. But for the sake of ready comparison of the results of the methods the different cements are arranged in the following group, according to the order of workability as indicated by the three methods:

Capillary tube	Ball plasticimeter*	Extrusion cylinder
12	9	2
9	2	12
5	6	9
1	5	8
4	1	1
7	4	10
11	10	7
2	3	5
3	8	11
10		6
6		4
8		3

* No tests made of Nos. 7, 11 and 12.

Résumé.—It would seem from these simple methods that it has been demonstrated that cements do differ markedly in their workable characteristics when mixed with water. It would seem quite logical to conclude that they would confer these different characteristics on any aggregate with which they might be mixed. To definitely establish this fact the work will be continued and extended to mortars and concretes. However, other apparatus will have to be used for these two products. In the meantime it is also intended to study further the characteristics of the pastes from cements. It is not felt that the studies so far made are other than very preliminary, but they have served their purpose in showing that pastes from different cements have different characteristics which can be readily demonstrated and are quite likely of importance in their effect on the workability of concrete.

There was presented in the early part of this paper a definition of workability as related to concrete. According to this, one of the characteristics of workable concrete is ready deformability. In discussing the data, obtained with the several pieces of apparatus, it has been indicated that the less readily deformed pastes would produce the most workable

concrete. It can be readily realized that a paste offering but little resistance to deformation would not have the quality of preventing the particles of aggregate coming into direct contact with one another when mixed with them. This lack of separation through lubrication on the part of a readily deformable paste would result in too great friction between the particles of aggregate during the handling of the concrete, with consequent difficult deformability and workability. The relation between the amount of energy required to deform a paste which would produce a satisfactory working concrete and the energy required to deform the concrete made with the paste is another matter for future work. However, we should not lose sight of the fact that a too easily deforming paste would not have the lubricating character or "body" required for the proper "lubrication" of the aggregate. Such a paste would give a "light" lubricant when a "heavy" one should be used.

Gradation and Character of Aggregates as a Factor in Workability.

By A. T. Goldbeck.*

Introduction.—Before attempting a discussion of the gradation and character of aggregates as a factor in the workability of concrete, it is necessary that we have clearly in mind what we mean by the term workability. Concrete is thought of as being workable when it may be conveyed with the greatest ease and with the least amount of segregation and when it may be distributed within the forms and brought to the desired finish with the least amount of effort. If conveyed by chutes the concrete must be mobile and capable of easy flow, yet must have unctuous properties which will hold its component particles together without separation of the mortar from the coarse aggregate. If buggies are used, the mobility of the concrete or the ease with which it will flow is not important in the transportation problem. If the plant is a central mixing plant and the concrete is transported in motor trucks, our principal concern is to prevent segregation during transportation. If wet concrete is used which will flow readily from the mixer into the truck and which is very mobile, we will find that when our so-called workable concrete has arrived at the end of its journey in the motor truck there will be a layer of wet mortar at the surface and the heavy coarse aggregate will have compacted at the bottom. A mixture having highly plastic properties but dry consistency is preferable when it is to be hauled by motor truck. Evidently, then, depending upon the mode of transportation of the concrete, we must have a sliding scale of workability. A mix, workable for one mode of transportation from mixer to forms, may be entirely unsuited and unworkable when other transportation means are used.

The workability of the concrete within the forms may also vary widely depending upon the peculiarities of the structure with which we are dealing. Thus, if we are depositing mass concrete, we need be little concerned with the difficulty of spreading the concrete within the forms, for little spreading need be done. We *are* concerned, however, with the economy of the mixture, with the production of concrete having correct properties for the structure such as strength, density, and perhaps impermeability but we also are concerned with the use of a mix having sufficient workability to produce the desired finish adjacent to the forms and without air-pockets within the mass. In this case considerations of economy of materials may be more important than the attainment of an easy flowing plastic mass. Here, large aggregates extending up to 6 in. in diameter and well graded to produce low voids are often used with economy.

Where sections are thin and are filled with reinforcing steel, the greatest degree of workability is required in the concrete mass. Especially is this so where there is little opportunity to assist the concrete in assum-

* Director, Bureau of Engineering, National Crushed Stone Association.

ing its final position. In concrete highway construction we speak of a concrete as being workable if it may be spread easily and if the finishing may be carried on with little necessity for the finishing machine to back up a number of times in order to attain the desired degree of smoothness. It is evident then that the degree of workability required is dependent upon:

(a) The method of transportation of the concrete.

(b) The character of the structure.

(c) The importance of workability in relation to other factors such as economy of mix, etc.

It seems necessary to express the above thoughts to make it clear that there may be other properties of concrete more important for the particular work in hand than the attainment of the highest degree of workability. Economy is overall cost and other properties desired in the finished structure often may best be obtained by purposely sacrificing some workability for more important considerations.

It will be assumed, however, that we are trying to produce a concrete having the greatest workability in the sense that it may be most easily handled and most easily finished after it is placed in position. The question of the gradation and shape of particles will be discussed from that standpoint. If by workability we mean mobility, and ease with which the concrete may be worked into position in the forms with no surface pockets and no large internal air voids or if we mean the ease with which the surface finish may be given to a structure such as a concrete road, then it would seem that we are dealing in large degree, although not wholly, with a question of internal friction within the concrete and those factors which make for low internal friction are those which will make for ease in handling. The following characteristics of both fine and coarse aggregates must be considered in their effect on the internal friction within a concrete mass:

1. Shape of particles.

2. Smoothness of surface or surface characteristics of the particles.

3. Mechanical grading of the particles from the coarsest to the finest.

(1) *Shape of Particles.*—Aggregates used in concrete involve two general shapes, rounded material such as gravel, and angular material such as crushed stone, crushed slag or gravel. The degree of roundness and the degree of angularity vary widely. If we conceive of the coarse aggregate as being in rather close contact it is not difficult to visualize the interlocked position taken by angular particles and it is not difficult to see that such particles must by virtue of that interlocking be retarded in their free movement more than is the case with rounded particles. Angular particles by their very nature will not permit of as free an internal movement as rounded particles and this is all the more accentuated in our present widely used, but highly unscientific, arbitrarily proportioned mixes, in

which the same proportions by volume are used for angular particles as for rounded particles.

Shape of particle does not alone control the mobility of the mass, for the coarse aggregate is lubricated by the mortar and almost equal mobility is possible of attainment whether rounded or angular coarse aggregate is used merely by a suitable change in proportions to accomplish that purpose. It must not be assumed, however, that such a change in proportions necessarily means the use of more cement in one case than in another to produce equal strengths, for it has been found that such is not always true.

The following tests made on extremely angular as against rounded coarse aggregate illustrate the above statement. In no case are they intended to be used as a general comparison of the relative merits of angular and rounded aggregates. They do, however, bring out certain important phases of concrete which should be understood in a discussion of workability:

MECHANICAL ANALYSIS OF COARSE AGGREGATE.

Sizes (Round openings)	Per cent Mixtures 1 and 2
2 : 1½	20.75
1½ : 1¼	10.38
1¼ : 1	10.38
1 : ¾	15.56
¾ : ⅝	10.38
⅝ : ½	6.60
½ : ¼	20.75
¼ : 0	5.20

The cement content was 6.3 bags per cubic yard of concrete for each of the following mixes:

Mix Number	Coarse Aggregate	Proportions	Modulus of Rupture 28 days	Water Ratio
1	Angular	1: 1.67: 3.5	698	0.76
2	Rounded	1: 1.36: 3.5	616	0.67

Mixtures 1 and 2 seemed to be equally workable as shown by the slump test but more effectively shown by the way the concrete could be placed in the forms in molding the test specimens. It will be noted, that, notwithstanding the higher water-ratio in Mixture 1 than in Mixture 2 the strength of 1 was highest. These results are not presented to indicate a general comparison of angular fragments with rounded fragments but merely to illustrate the point that it is quite possible to obtain equal workability in concrete having angular fragments in comparison with that having rounded fragments merely by increasing the sand content but without changing the cement content. The increased sand required a higher water-ratio in the angular aggregate concrete and yet the strength of this concrete was higher than that containing rounded coarse aggregate. This point is emphasized since it shows that the strength will not necessarily be lower in the angular aggregate concrete simply because it requires more

fine particles and more water than rounded aggregate concrete for equal workability. The water-ratio strength relation, after all, is only a general law, not applicable except under conditions which can be compared. In this particular case the aggregates were different and the law is seen not to apply.

As far as shape of coarse aggregate is concerned, the matter may be summed up by stating that angular fragments, for like volumetric concrete proportions, produce harsher working concrete than rounded fragments due, in part, to the interlocking of the coarse aggregate brought about by angularity but, in part also, to the fact that angular particles for like gradings have higher voids than rounded fragments. This being the case, if the proportions are stated in an arbitrary manner, and the same proportions by volume are used with both angular and rounded material, there will be the same mortar content in both cases, the rounded fragments will be more widely separated than the angular fragments and will be better lubricated by the mortar. Hence, the mass is more mobile. Increased workability, however, may often be obtained simply by the addition of extra sand to the mix without any sacrifice in strength and even with an increase in strength.

This point is well illustrated by tests reported by the U. S. Bureau of Public Roads [1], of which the following results are typical.

Mix	Size of Stone	Dry Sand Strength : Volume Pounds per sq. in.	Dry Sand Strength : Volume Pounds per cu. in.	Damp Sand Strength : Volume Pounds per sq. in.	Damp Sand Strength : Volume Pounds per cu. in.	Effect of Dry Sand Proportioning Increased Strength	Effect of Dry Sand Proportioning Increased Volume
1:2:4	¼ — 1½	2070	385	1850	360	11.9	6.9
1:2:3	¼ — 1½	2660	445	2780	410	—4.3	8.5
1:2:4	¾ — 1½	1800	374	1680	350	7.1	6.8
1:2:3	¾ — 1½	2310	435	2310	395	0	10.0

The above tests were made on concrete proportioned by volume, first with damp sand which was swollen due to moisture, then with dry sand. Naturally, more sand was used in the mixes proportioned with dry sand than with damp sand. Note that the increased amount of sand used in the 1: 2: 4 mixes actually increased the strength and also the yield. The increased sand in the 1: 2: 3 mixes was of no value or of slight detriment to the strength. The 1: 2: 3 mix proportioned with damp sand measurement had sufficient fine material, while the 1: 2: 4 mix did not have sufficient sand and was improved in workability, strength and yield by the addition of extra sand. In some cases perhaps it will be necessary to add extra cement also in order that the strength may not suffer through the extra sand addition, but the above results show that extra cement may not be at all necessary.

(2) *Effect of Surface Characteristics.*—Coarse aggregates have three general kinds of surface characteristics—extremely smooth as in most

[1] *Public Roads*, July, 1924, p. 21, "The Bulking of Moist Sands," by A. A. Levison.

water-worn gravel and in certain types of flint rock, rough as in most crushed stone, and extremely rough and pitted as in blast-furnace slag. These surface characteristics probably have two effects from the standpoint of workability. In the first place, roughness of surface produces surface voids in excess of the voids which would exist were the surfaces smooth. Hence, it should be expected that less workability will result in the rough surface aggregates due to their increased voids and the lesser quantity of mortar available for promoting workability. Secondly, roughness of surface probably makes for high internal friction in the concrete mass and therefore decreased workability. There do not seem to be any specific tests covering the effect of surface characteristics on workability but practice seems to point to the general truth that the rougher the aggregate, the less workable is the concrete when like volumetric proportions are used.

(3) *The Effect of Gradation of Aggregates.*—Undoubtedly, the most extensive work that has been done in the study of effect of gradation on the workability of concrete is that of the Structural Materials Research Laboratory of the Portland Cement Association. One of the most extensive series of the older investigations were those of Thompson and Fuller, reported in the 1907 Transactions of the American Society of Civil Engineers. As the result of this series of tests certain general conclusions were drawn by the authors as to the effect of gradation on the strength and workability of concrete. It is stated by them, "That with the same percentage of cement in a given volume of concrete, the densest mixture, irrespective of the relative proportions of the sand and stone, was in general the strongest."

"The Little Falls tests further indicated that, for the materials used, there was a certain mixture of sizes of grains of the aggregate which, with a given percentage, by weight, of cement, to the total aggregate, gave the highest breaking strength. In practice, also, it was found that the concrete made with this mixture worked most smoothly in placing."[2] Briefly, the authors further stated that, "The best mixture of cement and aggregate has a mechanical analysis curve[3] resembling a parabola, which is a combination of a curve approaching an ellipse for the sand portion and a tangent straight line for the stone portion. The ellipse runs to a diameter of one-tenth of the diameter of the maximum size of stone, and the stone from this point is uniformly graded."

According to this statement the best mechanical analysis for coarse aggregate is that which is known as a straight-line grading. For illustration, the following is a straight-line grading for stone extending from ⅛ in. to 2½ in. in size.

	Per cent
Passing ⅛-in. screen	0
Passing ¼-in. screen	5
Passing 1¼-in. screen	47
Passing 2½-in. screen	100

[2] 1907 *Transactions* of American Society of Civil Engineers, p. 67.
[3] Same as [2].

In the straight-line grading it is seen that the intermediate size screen has retained on it approximately 50 per cent of the total coarse aggregate. This grading gives a workable mix and also comparatively low voids. The sand portion, according to Thompson and Fuller, should have a maximum size of grain one-tenth that of the maximum size of coarse aggregate and the grading should be such that the combined cement and aggregate will follow the so-called ideal curve of grading. According to this law the sand portion should have a grading curve closely following an ellipse. For illustration, for a 2½-in. maximum size aggregate in which the proportions by weight are cement 12, sand 23.7, coarse aggregate 64.3, the so-called ideal mechanical analysis according to the theory would be as follows:

	Per cent
Passing ¼ in.	100
No. 4	86.5
8	60.3
16	42.2
30	25.3
50	12.7
100	0
F. M. =	3.73

Incidentally it will be noted that the ratio of sand to coarse aggregate in the above mixture (23.7: 64.3) is much smaller than generally thought desirable for good workability and the grading given in the above theory will be considered further in the light of more recent ideas.

The Fineness Modulus as a Measure of Workability.—The fineness modulus has been suggested as a means for determining the effect of the size and gradation of aggregate on the strength and workability of concrete. It is based on the mechanical analysis made with square opening sieves of the Tyler series. Fineness modulus is defined as the summation of the percentages, divided by 100, of the material retained on the following square opening sieves.

Sieve Number or Size of Sieve	Clear Opening
100	0.0059
50	0.0118
30	0.0236
16	0.047
8	0.094
4	0.188
⅜	0.375
¾	0.75
1½	1.5
3 in.	3.0

The fineness modulus is simply a single number which expresses the gradation of an aggregate and it has been found to be a very convenient method of calculating, in a preliminary way at least, the proper proportions of fine to coarse aggregate having given gradations. At the Struc-

tural Materials Research Laboratory a very large number of mixtures have been made up and ample opportunity has been had of correlating the strength and workability of the various mixes of concrete with the fineness moduli of the aggregates. As a result certain limits of fineness modulus have been stated by the Portland Cement Association as limits within which various concretes are workable, and beyond which they are harsh working. This is fully discussed in the literature of the Portland Cement Association and diagrams have been drawn showing the range in maximum permissible value of fineness modulus for various maximum sizes of aggregate and various average compression strengths with concrete of varying degrees of consistency or workability as defined by the standard slump test.

The range in values for maximum fineness modulus for proper workability when rounded aggregates are used is advocated as follows:

TABLE I.

Crushing Strength Pounds—28 Days	Slump Inches	Maximum Size	Range of Fineness Modulus Combined Aggregates
		3	6.6 — 7.0
		2	6.25 — 6.65
4000	½ to 10	1½	5.85 — 6.2
		1	5.45 — 5.85
		¾	5.10 — 5.45
		3	6.5 — 6.9
		2	6.15 — 6.55
3500	½ to 10	1½	5.75 — 6.10
		1	5.40 — 5.75
		¾	5.00 — 5.35
		3	6.42 — 6.8
		2	6.10 — 6.45
3000	½ to 10	1½	5.70 — 6.00
		1	5.30 — 5.60
		¾	4.95 — 5.30
		3	6.35 — 6.7
		2	5.95 — 6.3
2500	½ to 10	1½	5.60 — 5.9
		1	5.25 — 5.5
		¾	4.85 — 5.20
		3	6.25 — 6.55
		2	5.85 — 6.20
2000	½ to 10	1½	5.50 — 5.80
		1	5.10 — 5.40
		¾	4.75 — 5.10
		3	6.2 — 6.5
		2	5.7 — 6.1
1500	½ to 10	1½	5.4 — 5.7
		1	5.05 — 5.3
		¾	4.70 — 4.95

The table is intended to apply to rounded aggregates and it is stated that each value should be lowered 0.25 when other than rounded material is used such as crushed stone or slag, flat pebbles or stone screenings. This is equivalent to stating that such aggregates require in general somewhat smaller sizes of particles to give equal workability with the round particles. It is also stated that for work of massive proportions the fineness modulus may be increased 0.10 for ¾-in., 0.20 for 1½-in., 0.30 for 3-in. and 0.40 for 6-in. aggregates. However, since the above figures are based on *average* conditions only, they do not necessarily apply to individual cases. They may often be and should be exceeded for the most economical results but they do serve as an excellent guide in determining the probable workability of a given combination of aggregates.

The following facts are indicated by the above average table with regard to the gradation of aggregates:

(1) The stiffer the mix the smaller should be the aggregates for equal workability.

(2) The higher the strength, the coarser may be the aggregate having a given maximum size. This is possible undoubtedly because of the use of more cement containing fine particles which lubricate the mass.

(3) It is possible to obtain a sufficient degree of workability irrespective of the maximum size and irrespective of the aggregate characteristics by changing the concrete proportions.

It is apparent that since a number of gradations may result in the same fineness modulus and, further, since the gradation affects the voids in the coarse aggregate, it is quite possible to have different degrees of workability in concrete in which the coarse aggregate has the same fineness modulus. The following table made up from results obtained in the Kansas State Laboratory illustrates this point.[4]

Batch No.	F. M. of Coarse Aggregate	F. M. of Mixed Aggregate	Voids in Coarse Aggregate
5	7.55	5.93	48.0
7	7.30	5.76	42.0

The fineness moduli of the coarse aggregates in batches Nos. 7 and 5 are almost equal. The voids, however, are sufficiently different to make a difference in workability when like concrete proportions are used. This is so because the mortar has to fill 6 per cent more coarse aggregate voids in batch No. 5 than in batch No. 7 and is, therefore, not so effective in separating the particles of the coarse aggregate. The coarse aggregate in batch No. 7 was well graded from fine to coarse; that in No. 5 was of more nearly uniform size. Batch No. 5 needs a higher proportion of finer particles than batch No. 7 to produce equal workability in both cases but

[4] "Cement Requirements for Some Concrete Mixes," by Harold Allen, Assistant Engineer of Tests, Kansas State Testing Laboratory. *Concrete,* Nov., 1927, p. 13.

this does not necessarily mean that the same cement-sand ratio must be used in both cases to obtain equal strength.

Before leaving the subject of fineness modulus it will be interesting to determine the fineness modulus of the aggregate graded according to Thompson and Fuller's curve which has been stated to give the greatest workability. The following is an ideal gradation according to the curve for 2½-in. maximum size and 12 per cent by weight of cement.

Sieve No	Per cent of Cement and Aggregates Retained on				Per cent of Total Aggregate Retained on
100	88	x	100/88	=	100
50	85				97
30	82				93
16	78				88
8	74				84
4	67				76
⅜	61				69
¾	50				57
1½	29				33
		F. M.		=	6.97

For 1½-in. maximum size aggregate the highest permissible fineness modulus is 6.2 for 4,000-lb. concrete and 5.7 for 1,500-lb. concrete. Apparently then, according to the fineness modulus method of estimating the probability of workability, aggregate graded according to the Fuller-Thompson curve is too coarse for workability and should contain more fine material.

What is the Best Gradation of Coarse Aggregate for Workability?—Investigations aimed at an answer to this specific question are not easy to find and it is questionable if a very specific answer can be given in the absence of extensive test information. However, there are certain indications which permit of a tentative answer.

It seems to be the case that the smaller the maximum size of coarse aggregate, the greater is the workability and also the more uniformly graded the coarse aggregate from maximum to minimum size, the more workable is the mix. The minimum size, for commercial reasons is generally ¼-in. and sand is rather universally allowed to extend up to ¼-in. in diameter. It is undesirable that the coarse aggregate have too high a proportion of particles between ¼- and ⅛-in. in size, especially when the sand is coarse, for a "grainy" concrete will result which will not work smoothly. A straight-line grading for coarse aggregate gives good workability, economically. Perhaps this grading may be slightly improved by decreasing the percentage of medium size and increasing the percentage of smaller sizes slightly. It is well to limit the per cent passing the ¼ to 10 per cent and preferably 5 per cent to avoid "grainy" concrete and there should be approximately 50 per cent retained on the screen midway between the ¼-in. and the maximum size.

Gradation of Fine Aggregate for Workability.—In general, the finer the fine aggregate, the greater will be the workability. This does not mean the use of a fine sand is desirable, for considerations such as strength and economy require the use of as coarse a fine aggregate as will make workable concrete. A sand may produce harsh working concrete if it has too high a percentage of particles above the No. 8 sieve, especially if there is a large percentage of small particles in the coarse aggregate. It is almost impossible to set down any hard and fast rule for limitations in the gradation of sand from the standpoint of workability. The fine and coarse aggregate combinations must be considered together.

Merely as a guide, subject to actual trial mixes, it is suggested that the limiting values for fineness modulus of the combined fine and coarse aggregate given in Table II be used to determine whether a mix containing any particular sand and coarse aggregate will be workable. The indications from the Structural Materials Research Laboratory tests are that a

TABLE II

Mix, Cement-Aggregate	Size of Aggregate													
	0–No. 28	0–No. 14	0–No. 8	0–No. 4	0–No. 3*	0–3/8 in.	0–1/2* in.	0–3/4 in.	0–1* in.	0–1 1/2 in.	0–2.1* in.	0–3 in.	0–3 1/2* in.	0–6 in.
1:9.......	1.30	1.85	2.45	3.05	3.45	3.85	4.25	4.65	5.00	5.40	5.80	6.25	6.65	7.05
1:7.......	1.40	1.95	2.55	3.20	3.55	3.95	4.35	4.75	5.15	5.55	5.95	6.40	6.80	7.20
1:6.......	1.50	2.05	2.65	3.30	3.65	4.05	4.45	4.85	5.25	5.65	6.05	6.50	6.90	7.30
1:5.......	1.60	2.15	2.75	3.45	3.80	4.20	4.60	5.00	5.40	5.80	6.20	6.60	7.00	7.45
1:4.......	1.70	2.30	2.90	3.60	4.00	4.40	4.80	5.20	5.60	6.00	6.40	6.85	7.25	7.65
1:3.......	1.85	2.50	3.10	3.90	4.30	4.70	5.10	5.50	5.90	6.30	6.70	7.15	7.55	8.00
1:2.......	2.00	2.70	3.40	4.20	4.60	5.05	5.45	5.90	6.30	6.70	7.10	7.55	7.95	8.40

* Considered as "half-size" sieves; not used in computing fineness modulus.

fineness modulus of 3.50 is about the maximum permissible unless the sand extends up to 3/8-in. in diameter when 3.75 may be permitted. A rule suggested in Bulletin No. 1 of the Structural Materials Research Laboratory is that "Sand or screenings used for fine aggregate in concrete must not have a higher fineness modulus than that permitted for mortars of the same mix." Thus, in the above table from Bulletin 1, if the ratio of cement to total aggregate of 1 1/2-in. maximum size is 1: 5 (about a 1: 2: 4 mix) the fineness modulus of the combined aggregate should not exceed 5.80 and that of the fine aggregate of No. 4 maximum size should not exceed 3.45. The above rule is about the most definite which can be stated as to the grading of fine aggregates which will make workable concrete. From the standpoint of economy it is well to use a sand with as high a fineness modulus as possible within the limits determined as above.

As an illustration, of very desirable gradings for sand, the following are given. These gradings closely approach the elliptical gradings required for good results by Thompson and Fuller's curve and, moreover,

have desirable values for fineness modulus for the average run of concrete mixes:

Size of Sieve Tyler Series	Percentage by Weight Retained
⅜	0 to 5
No. 4	0 to 10
8	10 to 35
14	24 to 45
28	46 to 68
50	70 to 95
100	94 to 100
	Fineness modulus....2.44 to 3.58

Conclusions.—In conclusion the following short statements may be made:

(1) When like proportions by volume are used, angular, rough aggregate makes a harsher working concrete than round, smooth aggregate due:

a. To difference in voids and the resulting variable effect from the mortar in producing workability.

b. To increased internal friction due to the angular shape and rough faces of the particles.

The relative influence of these effects has not been evaluated. If like workability is to be expected from the different aggregates, the illogical method of using the same arbitrary proportions by volume should not be used. Harsher aggregates require more fine material in the mix but not necessarily more cement to produce concrete of the same workability as rounded fragments. The practice of using the same arbitrary proportions for all aggregates cannot be too strongly condemned.

(2) A straight-line grading for the coarse aggregate extending from the maximum size to 1/10 of the maximum size provides a well-graded stone from the standpoint of workability. The difficulty of using such a grading without proper precautions to prevent segregation is recognized. The gradation for given maximum sizes should be such that the fineness moduli of the combined aggregates given in Tables I or II are not exceeded.

(3) The gradation of the fine aggregate should be as coarse as possible for economy but should not result in a fineness modulus in excess of that stated in Table II for a mortar having the same proportions as the particular concrete in which the sand is to be used.

Water as a Factor in Workability.

By R. L. Bertin.*

In discussing the effects of water or any other ingredients used in making concrete on the workability of concrete, it is, I believe, important first to establish what is meant by workability. In so far as workability applies to concrete, I take it to be that property by virtue of which it may be properly worked in place with ease, with the least amount of energy, in the least time, therefore at the least cost.

It stands to reason that the placing of concrete must be done in such a way so as not to impair the requisite qualities of the finished concrete structure. The requisite qualities of concrete may be divided into two classifications:

1. Those which are general to all concrete.
2. Those which are specific for special purposes.

Under the first classification, I include homogeneity or uniformity, and under the second, I include strength and density. I exclude from consideration, for the purpose of this discussion, the qualities which depend on the properties of the materials themselves: resistance to chemical action, heat, cold, impact and abrasion. Concrete, no matter what its service might be, should be uniform, and uniformity means that the various ingredients should be evenly distributed throughout the mass.

Strength and density requirements of concrete may vary in degree, depending upon the purpose for which the concrete is used, but such degree as the designer assumes in his design must be obtained. In varying the workability of concrete with the addition of water, the amount of water is restricted to that which will not cause the materials to segregate and will produce a concrete of sufficient strength and density.

In order to understand clearly the effect of more or less water on concrete during the making and placing period, it is best to follow, step by step, the changes which take place in a concrete having a given proportion of dry cement, sand and stone, to which water is added gradually, from the dry state to the very wet. With the first increments of water, the dry mass bulks. This bulking increases to a maximum, depending upon the nature of the materials used; then, as more water is added the bulking gradually decreases until a maximum density of the mass is reached. At that point the concrete becomes plastic. The addition of more water causes the mass again to increase in volume until a point is reached when segregation takes place because of the dilution of the mixture. The minimum amount of water which should be used is that which will overcome the bulking of the mass, and the maximum, that which will produce concrete of the required strength and density and which will not start segregation. These maxima and minima vary with the nature of the materials used and

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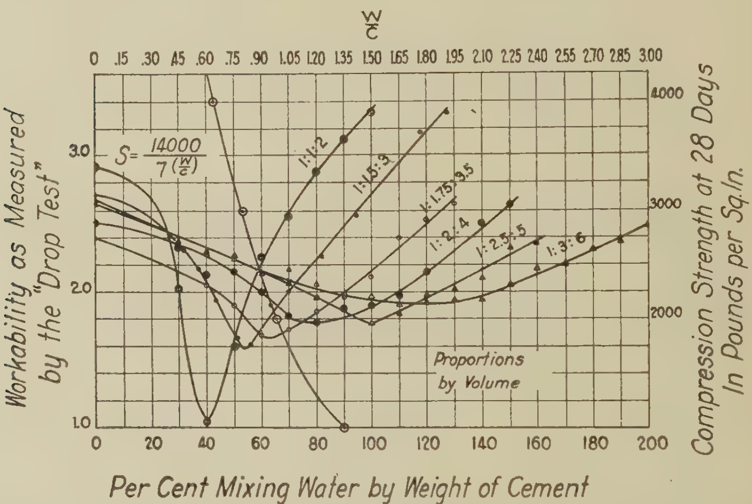
their proportion, and the required strength and density of the finished concrete.

The limitations of water content mentioned are the ones within which workability may be varied with the use of more or less water. If proper workability cannot be obtained within these limits, the concrete proportions or materials must be changed.

The effects of water on workability (the only influences here considered) within the entire range of water content for varying mixtures have been investigated by Tokujiro Yoshida in connection with his drop device



RELATION BETWEEN WORKABILITY AS MEASURED WITH YOSHIDA DROP DEVICE AND PERCENTAGE OF WATER BY WEIGHT.

for determining the workability of concrete,[1] and I have reproduced one of his diagrams showing the relation between workability as measured with his apparatus and the percentage of water by weight of cement. This expression bears a direct relation to the water-cement ratio, and I have added to his diagram the water-cement ratio corresponding to the water percentages. I have also shown on this same diagram the water-cement strength relation

$$S = \frac{14,000}{7^{\left(\frac{W}{C}\right)}}$$

From this diagram, the following deductions may be made:

1. The richer the mixture the greater is the variation in workability obtained with a given amount of water.

[1] A new Test Method for Workability of Concrete by Tokujiro Yoshida, A. C. I. *Proceedings*, Vol. 23, page 415.

2. The workability of lean mixtures is not greatly benefited by the addition of a considerable amount of water.

3. For a given mixture, the workability increases in about direct proportion to the water-cement ratio.

Mr. Yoshida states in his paper that the minimum workability is obtained when a sufficient amount of water is used to produce concrete of maximum density. The minimum points on the curves therefore represent the percentage of water necessary to render the concrete plastic and represents the minimum amount of water which should be used for any of the mixtures shown on the diagram with the aggregate used, namely:

Sand having a F. M. of 3.54
Stone " " " " 6.92

That part of the curves from 0 per cent of water to the point of minimum workability is not to be considered, as the concrete it represents has an insufficient amount of water to make it plastic and therefore uniform. That part of the curves starting from the minimum workability point on is the practical range which may be used. This range is limited by the strength or density required; the strength curve shown on this diagram enables one to establish this limit. If the curves were carried only to that water content where segregation of the materials begins, the other limitation would be apparent.

With these limitations in view, it may be seen that the variation of workability, which may be obtained by the addition of more or less water except for the very rich mixture, is limited. Any one with simple appliances can conduct a series of tests with a variety of aggregates mixed in different proportions, plot the results as shown on this diagram and learn much regarding the effects of water on workability within the limits imposed by the requirements of the concrete one wishes to produce.

WORKABILITY MEANS DURABILITY TO THE ENGINEER.

BY R. W. ATWATER.*

The engineer is traditionally at a disadvantage in dealing with concrete as a construction material. His work involves selecting suitable materials and combining and shaping them to serve his purpose. While he is free to use originality in the selection of materials and the manner in which he assembles them, still the essential materials with which he works, with the exception of concrete, come to him as definite entities and are identified by their characteristics and qualities. The location of the project does not usually affect the selection of the material. Whether a structure is to be erected in New York or San Francisco, if the steel for it is specified to meet the A. S. T. M. Standard Specifications for Structural Steel for Buildings, the steel that is furnished in both cities will be of equal strength and quality within permissible tolerances. Thus the engineer is accustomed to selecting materials for their recognized characteristics and qualities and writing his specifications accordingly. Before the materials are incorporated in the structure and usually before they are delivered to the work, they are inspected and tested, and accepted or rejected according to whether or not they comply with the engineer's specifications and the standard specifications for that class of material.

The engineer usually has but little concern in the manufacture of these materials. While he may have some knowledge of the composition of the material and process of manufacture, there is no occasion for him to have contact with its manufacture other than the usual inspection and testing. To successfully design and supervise the construction of a steel structure, he need not qualify as an expert in the chemistry and technology of steel making, or as a rolling mill, fabricating shop and erection superintendent. Manufacturing and testing methods have been sufficiently standardized so that he can confine his interest to the finished product. If the material submitted does not comply with the specification it may be rejected and additional material submitted until it is furnished to meet the requirements as specified.

Concrete, however, cannot be considered in the same sense or handled in the manner as other construction materials. It is uncertain whether concrete ever becomes a finished product. It does not possess qualities of hardness and strength until after it has been placed in the forms and has attained its initial set, and some time must elapse thereafter, according to the cement used and the method of manufacture, before these qualities develop to a degree to be useful. Even after this the cement reactions may continue for an indeterminate period with the character of the material changing accordingly. After it is a part of the structure any tests which might be made to ascertain its strength and qualities are only of historic value. Since concrete comes to the work not as a finished product,

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but in the form of cement and aggregates, there can be no test and rejection process applied to it as a finished product in advance of its becoming part of the structure to assure that it possesses the desired qualities. In fact, there can be no standard established by which to judge its composition and quality, because of the variation that is encountered in the characteristics of the constituent materials in various localities, and because of the present lack of standardization in the methods of manufacture.

Concrete is a combination of water, cement and aggregates in a mixture and its composition and qualities may be affected by any variation in any of its constituents or in its manufacture. Since the composition and fineness of cement, and the character, size and grading of aggregates vary widely according to the source of supply, it is not to be expected that concrete in one locality should have the same structure and composition as in another. Concrete is manufactured at the work and its strength and quality of necessity must be predetermined, and the selection of materials and the process of manufacture must be regulated to produce the desired results.

The engineer's and contractor's organizations jointly compose the personnel which manufactures concrete. The engineer lays out and directs the operation and the contractor performs the work. The engineer, according to the requirements of his design, determines what the strength of the concrete should be and what special qualities the concrete should possess other than being a hard durable material, such as water-tightness, resistance to abrasion, resistance to sea water and resistance to acid or alkali solutions.

The materials to be used and the process of manufacture are stated in the engineer's specification for the guidance of the contractor, whose organization performs the work accordingly, but it is essential that the engineer's organization should function throughout the work, especially at the start of the operation, to determine if the work can proceed efficiently and economically, as outlined in the specification, and to make the necessary corrections in the procedure should this not be the case; also to require strict observance of the specification throughout the work and to make such further amendments in the outlined procedure as may be necessary as the work progresses.

Thus, it is essential that an engineer who is to successfully design and supervise the construction of structures involving concrete should be thoroughly informed concerning the technology and manufacture of concrete. The strength and quality of concrete reflects in a large measure the degree of knowledge and effort exerted in its manufacture by the engineer in charge of the work. To some engineers this condition has long been apparent, but to the many it is somewhat new and startling, for they have been in the habit of regarding concrete much the same as the other materials with which they deal. They have expected concrete to have certain definite characteristics and qualities and when the material was found to be lacking in these respects keen disappointment and regret were expressed,

and the material criticized for not being suitable for the purpose. Their practice has been to specify concrete on the "arbitrary proportion" basis, and after being assured that clean, sound aggregates were supplied they have permitted the contractor to mix and place the material, allowing the water in the mix to be controlled to suit the method of placing. That this procedure has resulted in a large amount of unsatisfactory concrete is not a reflection upon the integrity or ability of the contractor, for the reason that his relation to the work does not qualify him to accept responsibility for the quality of the concrete and he should not be expected to assume this liability, which is primarily not his, but that of the engineer. On the other hand the engineer cannot be duly criticized for not recognizing earlier his true responsibility in the manufacture of concrete. The fallacy of the practice formerly in vogue was not generally recognized until more reliable information bearing on the composition and manufacture of concrete became available through research work which has been conducted during the past few years.

The lack of knowledge that existed concerning concrete is but another indication of the difference between concrete and other materials of construction. Materials such as steel and brick are manufactured by corporations who singly or in combination have devoted funds to the establishment of laboratories and the employment of scientists for the purpose of improving their materials. Competition between manufacturers has stimulated this work. The research work that has been done in the field of concrete has been voluntary to a large degree, and has lacked the direction and co-ordination that would exist if the manufacture of concrete was more centralized. In spite of this, substantial progress has been made, and while there is still much to be learned about the chemistry of concrete, it is quite well understood what results may be expected when water, cement and aggregates are combined under definite conditions. This information has been obtained by what may be termed the cut and try method, that is, by making many thousands of specimens and by testing and observing their characteristics. From the uniformity of results obtained it is generally conceded that there are certain operations in the manufacture of concrete which have definite influences upon the strength and quality of the resulting material.

If the aggregates are clean and sound, and the cement of suitable quality, the strength of concrete can be controlled by the amount of water that is added to a given volume of cement in the mix, provided the mix is workable. This is known as the law of water-cement ratio, and by its use the strength of the concrete can be controlled with accuracy heretofore considered impracticable. The temperature at which concrete is mixed, placed and cured is of vital importance in the strength and quality of the resulting material. The curing of concrete, that is, the supplying of moisture to the surface of concrete during a considerable period after the initial set, has an influence on the surface condition, and on the completion of the cement reactions which in turn favorably affects the durability and wear-

ing qualities of concrete. But the feature that appears to be the most important in the production of durable concrete is the proper control of the workability of the mix.

Workability is the state of plasticity of the concrete during its manufacture. Concrete to be of maximum durability should be mixed so that in its plastic state it will not be so dry as to crumble, and not so wet that there appears to be any indication of segregation when the concrete is handled. In the manufacture of concrete workability may be used as a predetermining factor of its durability and permanence. If the mix is proportioned and mixed to the correct degree of workability there will be minimum of segregation when the material is placed in the forms and puddled into the angles and corners and around the reinforcing steel, and thus the concrete will be of maximum uniform structure throughout its mass. There will be a minimum of water gain as the material is placed and a minimum of separation of fines which when the mix is too wet rise to the top of the pour and form laitance. There will be a minimum of honeycombing both external and internal and the concrete will have the maximum density which can be obtained using the materials available. This maximum density assures minimum porosity and permeability and maximum water-tightness. So long as water or other corrosive agencies can be excluded from concrete its durability is assured. Disintegration and deterioration of concrete are usually due to the mechanical action of frost spalling off portions of the material or the breaking down of the chemical structure of the concrete by later chemical reactions caused by water or other corrosive agents entering the concrete due to its porosity or permeability.

Workability is probably the most difficult factor to control in the manufacture of concrete. Workability is affected by the quality and fineness of the cement, the character, size and grading of the aggregates, the quantity of water in the mix, the type of mixer and the time of mixing. There is also a tendency to make workability subordinate to the methods of placing. Because workability has a bearing on the method of placing and fixes the proportions of cement and aggregate in the mix, it is one of the controlling factors in the cost of concrete.

Contractors who have been in the habit of handling concrete through long lines of chutes and spading it from single points of deposit to the remote corners of the forms are likely to complain that concrete of the correct degree of workability is expensive to handle, and that this requirement increases the cost of the work. On the contrary, it has been found in practice that, when concrete is mixed to the plasticity which gives minimum segregation, it flows readily and can be easily puddled into all corners and angles of the forms and around the reinforcing steel. It may not flow as readily in chutes as a wetter mix but after the placing methods are modified to handle this type of concrete, it is usually found that the placing cost does not exceed that of the wetter mix. On the other hand, the cost of materials mixed to the proper degree of plasticity is less than for

a wetter mix, provided the water-cement ratio is maintained. This naturally follows because the dryer mix contains more aggregate per volume of cement than a wet mix and as cement is the most costly ingredient in concrete, the less cement that is used per unit volume, the lower the unit cost for materials. Workability has another influence on the cost of the work in that the amount of rubbing, patching and finishing required to give acceptable appearance is less for concrete which is mixed to the right degree of plasticity than for a wetter or dryer mix.

The compressive strength of concrete as indicated by test specimens crushed at 7 or 28 days cannot be used as a measure of the durability of the concrete of which these specimens are representative. The mix may be designed for a predetermined strength and so long as the aggregates are sound and clean and the water-cement ratio is accurately maintained, the strength of the concrete as indicated by the 7 or 28-day tests will be constant within permissible limits regardless of the degree of workability, provided the mix is not too harsh. A rule might be formulated on the basis of certain observations that concrete with a compressive strength of 2,500 lb. per sq. in. at 28 days will have maximum durability. However, a recommendation of this sort has no value, for though the cylinder tests might indicate the concrete was of this compressive strength, still the mix might be so wet that the concrete would not be of maximum durability. If the mix is too wet, durability is not only sacrificed but the assurance, given by the reports of the compression tests, that the concrete is of the required strength will probably be misleading. Through the action of gravity, concrete that is mixed with too much water assumes when placed a formation that varies in density from the bottom to the top. The water and fines separate from the heavier aggregate which settles to the bottom and causes the lower part of the pour to be harsh, that is, to have too little water and cement. The upper part of the pour becomes laitance underlaid by a grout of uncertain value. Thus the control of the water-cement ratio, which may have been accurately maintained at the mixer, is completely unbalanced in the placing and the concrete in the structure is not only not durable but it is not uniformly of the required strength.

Another serious consequence of segregation and the resulting depositing of a harsh mix at the bottom of the pour is that the aggregate may arch over the reinforcing steel and thus form pockets and voids which will prevent bond on the bars and reduce resistance to shearing stresses. This condition can obtain in a large footing or girder and escape detection until failure results.

Whenever concrete is to be used in a location where the exposure to an active destructive agent, such as sea water, is severe, extraordinary precautions should be taken to ascertain the water-cement ratio and the degree of workability that will give a concrete of maximum resistance to deterioration under exposure to this agent. Theories and formulas which give good results in one location should never be used in another unless thoroughly verified by preliminary tests and observations, using cement

and aggregates available in the location where the work is to be done. Usually the nature of the work does not permit the making of specimens representative of various water-cement ratios and degrees of workability and submitting them to prolonged exposure to the destructive agent. If such a procedure is practical, such observations will be of great value in determining the proper mix. Whether or not these prolonged tests can be made, the true characteristics of concrete made from the available materials should be ascertained. Specimens representative of all the cement and aggregates available, and of a wide variation of water-cement ratios and degrees of workability should be made and tested for compressive strength, absorption, permeability, density and percentage of uncombined water. The mix to be used for the work should be one that gives maximum density and minimum permeability, absorption and percentage of uncombined water.

The results of these tests may indicate that even with accurate control of the relative consistency and workability of the mix, concrete cannot be economically manufactured from the materials available that will give sufficient resistance to the destructive agent present to be durable. In this event experiments should be conducted using various types of admixtures to ascertain if their use in the mix will decrease the liability of deterioration. Another possible method of procedure is to design the mix for the maximum results that may be obtained from the available materials and to coat or impregnate the surface of the concrete with some material that will repel the destructive agent or prevent it from entering the concrete. Both of these methods should be thoroughly investigated and their beneficial effects verified by test before either is adopted.

The general acceptance of the importance of controlling workability has been accompanied by the marketing of a large number of admixtures in liquid or powder form. These admixtures are recommended to have various beneficial effects on concrete, such as to improve workability and thus durability, to make the concrete water-tight and to improve the yield of the mix and thus improve economy. These admixtures undoubtedly have a place in the manufacture of concrete and can be advantageously used under certain conditions. However, they should not be used indiscriminately. The possibilities of making a satisfactory concrete out of the cement and aggregates available should be thoroughly disposed of before the use of admixtures is considered, unless it is very apparent that their use has a decidedly beneficial effect on the quality or economy of the work.

The elaborateness of the engineer's specifications and field supervision is naturally governed by the magnitude and importance of the project at hand. If the project is relatively small and unimportant, the proper relative consistency for the desired strength can be obtained from the curves worked out by Professor Abrams and the proper degree of workability arrived at by making successive batches using various proportions of fine and coarse aggregates and observing the characteristics of the mixed material; but even for small projects, the value of field supervision should not

be underestimated. The total water in the mix and the aggregates should be accurately measured and corrections made for the moisture in the aggregates. A mix of the proper degree of workability can be handled efficiently and economically if successive batches are uniform, and uniformity can only be maintained by accurately measuring the water and aggregates. As the project increases in size and importance, the necessity for accurate measurement increases accordingly. The amount that can be expended for plant on any job is a function of the amount of yardage. The engineer should keep this constantly in mind and when the work is of sufficient importance and involves sufficient yardage, he should specify and require the use of measuring and mixing devices which are most likely to produce the desired results. This should not be left to the discretion of the contractor. As the yardage increases, there is a tendency to reduce the time of mixing. The time of mixing as well as the type and size of mixer has a definite influence on workability and the number and type of mixers used on the work and the time of mixing is not the contractor's prerogative but should be definitely covered in the engineer's specifications and his field supervision should be especially alert in this respect.

Workability is the measure which can now best be applied in the manufacture of concrete to predetermine the quality of the resulting material. Subsequent research may show that the criterion for maximum durability and permanence is some factor altogether different from workability and not known or recognized at present. Whatever the future has in store in this respect will be gratefully received and the process of manufacturing concrete modified accordingly, but until that time the value of workability should be recognized and accepted and the engineer should give it proper consideration in his specifications and field supervision.

Much unsatisfactory concrete has resulted and will continue to result, not because the engineer in charge of the work is not qualified or because his preliminary investigations and specifications are not adequate, but because the field supervision as performed by the members of his organization is not commensurate with his own efforts. The men chosen to supervise the manufacture and placing of concrete should be thoroughly qualified by education and experience to do this class of work.

The attitude of the contractor will have a material bearing on the efficiency, economy and results obtained. Many contractors are now thoroughly alive to the possibilities of manufacturing better concrete and their co-operation is assured on any work with which they are connected. Others are not so willing to relinquish what has been assumed to be their prerogative and to adopt methods which are consistent with proper workability and which guarantee accurate measurements and uniform batches. The proper control of the manufacture of concrete is made doubly hard for the engineer's organization when dealing with a contractor of this type and until he sees the light and indicates the desire to offer facilities and co-operation in the manufacture of good concrete it will be found that the engineers and contractors who are co-operating in this respect will receive preference especially on the more important work.

What Workability Means to the Contractor.

By Nelson L. Doe.*

During the past five years remarkable progress in the making and placing of concrete has been made. The contractor of today realizes that a properly designed mix which will produce workable concrete is necessary. Furthermore, the up-to-date contractor is well aware that workable concrete is much more economical, as to cost of materials and expense of handling than the average concrete made without a calculated mix.

Variations in theories of design, the range of aggregates, the different requirements of structures in which concrete is used, the extensive array of modern field equipment available, and the difference in labor markets, introduce so many variables that the production of a workable concrete with maximum economy becomes a study worthy of considerable attention.

To produce a concrete of given strength, having maximum workability at minimum expense, the cost of materials and labor must be properly proportioned.

Workability as Affected by Materials.

Considering first the cost of increasing workability by altering the material content of the concrete, we find that the following possibilities are usually present:

(1) The addition of more cement at the same time adding sufficient water to keep the water cement ratio a constant.

(2) A variation in the amount and grading of the aggregate.

(3) The use of an admixture.

Workability of concrete for complicated portions of structures is commonly increased by the addition of more cement than the specifications require. This practice is probably true economy. There is no question as to the effect of extra cement. The workability increases as the amount of cement is increased and the quality of the concrete is improved.

Whenever additional cement and water are added for the purpose of increasing workability, due care must be used to see that the water cement ratio is not increased; otherwise a decreased final strength will result. By mixing a few trial batches containing various amounts of additional cement, each batch being made with the correct water cement ratio, the effect of extra cement on the quality of the concrete can be observed, and the increased cost of obtaining better workability by this method can be determined.

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In cases where it is impossible to alter the grading of the aggregates, additional cement and water will give increased workability without excessively increasing the cost of the concrete. In the average situation, however, sufficient workability can be secured in more economical ways.

The second possibility for increasing the workability of concrete by variations in the aggregates is probably the most attractive and interesting study from an economic standpoint. The fact that all aggregates having the same fineness modulus produce concrete of the same strength, other conditions being identical, has been quite generally established and accepted. Perhaps it has not been as fully emphasized that different aggregates having the same fineness modulus do not produce concrete of the same workability, or yield the same volume. These features are important from the contractor's point of view, and present opportunities for increasing workability and yield with no sacrifice in final strength, by a re-grading or re-selection of aggregate. It is not unusual to find different aggregates in the same market sold for approximately the same prices that produce concretes in which the cement factors vary from 10 to 25 per cent with a corresponding range in workability. This variation of yield and workability, due to the grading of aggregates, applies to both large and small materials. Poorly graded crushed stone will always give less workability and a smaller yield than a well-graded gravel of the same fineness modulus. Likewise, bank sand running heavy in fine materials will show better results than a sand taken from water, which is apt to be lacking in fines. In studying the effect of sands upon workability, the percentage of material passing the No. 28 and 48 sieves is very important. Sands to give workable mixes should have a sufficient percentage of suitable materials passing these sieves. Sands that do not contain sufficient fine materials give harsh mixtures and show excessive cement factors. To illustrate this point, it is interesting to compare two sands sold in the New York market for the same price: Sand No. 1 has a fineness modulus of 2.46, with 59 per cent passing the No. 28 sieve, and 19 per cent passing the No. 48 sieve. Sand No. 2 has a fineness modulus of 2.80 with 44 per cent passing the No. 28 sieve, and but 6 per cent passing the No. 48 sieve, according to the grading tabulated below:

Sieve No.	Per cent coarser than sieve. Sand No. 1	Sand No. 2
4	0.6	0.6
8	5.6	6.8
14	17.4	22.4
28	41.2	56.2
48	81.2	93.8
100	100.0	100.0
	246.0	279.8
Fineness modulus	2.46	2.80

The final records of one New York contractor, covering two similar building operations, involving 57,000 yd. of 2,000-lb. concrete, placed by the same field force under similar conditions and with the same coarse aggregate, but with sand No. 1 used on one project, and sand No. 2 on the other, show a cement factor of 1.24 for the finer sand compared with a factor of 1.48 for the coarser sand. In other words, one more bag of cement per yard of concrete was used with the No. 2 sand. These records seem to bear out experiments on yield values and show to the contractor the importance of obtaining properly graded materials.

In any locality, then, the first effort toward securing workability in an economical manner should be to intelligently select the aggregates. A careful sieve analysis of those available should be made, and actual test yields compared. Unless a proper combination of aggregates is selected, maximum economy and workability cannot be expected.

The third way in which workability can be altered by changing the material content is by the use of an admixture, such as lime or celite. Neither of these admixtures, as commonly used, seems to affect the final strength of the concrete to any extent. Both materials increase the workability, apparently in the same manner that the addition of very fine aggregates or extra portland cement increases workability. Both cost considerably more than portland cement, and the handling of the admixture involves another operation at the mixer. For ordinary concrete work, it will usually prove cheaper to increase the workability by a change in the aggregate or by the use of additional cement, or both, than it will to use an admixture. Under conditions where exceptional workability is necessary, in isolated locations where transportation of cement is costly, or where properly graded aggregates cannot be obtained, celite or lime may work out as a distinct advantage. When considering the cost of admixtures, the bulking effect of celite on certain concretes should not be neglected.

WORKABILITY AS AFFECTED BY LABOR.

The quality of workability bears a very close relation to the handling and placing operations wherever concrete is used. The most important features in the field which affect workability are as follows:

(1) The plant layout for handling and measuring materials before they enter the mixer.
(2) Transportation methods and means.
(3) Care and system used in placing concrete in the forms.
(4) General control and regulation of field operations.

The type of equipment used for mixing concrete has changed very little since the first Ransome mixer was built. It can be safely said that the majority of mixers as designed today are satisfactory and will produce a thoroughly mixed concrete if properly handled. The principal changes that have been made in equipment, tending toward more uniform work-

ability, are in the devices for measuring or weighing either the cement, aggregates, or the water. On many modern plants, sand inundators, automatic water tanks, and hoppers for accurately measuring or weighing aggregates have been installed and successfully operated. Aggregates must be measured accurately and each batch must contain the correct amount of water, if uniform workability is to be secured. There are, however, economic limits to this accuracy of measurement which must be considered when measuring or weighing equipment is selected.

The cost of installation per yard of concrete is normally higher for smaller operations, than where large volumes of concrete are handled. The cost of operating elaborate plant installations also runs heavy per yard of concrete unless handled very efficiently. Frequently also conditions at the site prevent the installation of plants requiring considerable head room.

For such reasons, it is often sound economy to knowingly sacrifice sufficient additional cement to allow for reasonable variations in measurements, and to install a simple plant without precise measuring devices. For instance, a labor and material charge for special plant items, installation and increased operating expenses amounting to 50¢ per yard, would cost the contractor about the same as though an extra bag of cement had been added to each yard of concrete. The additional bag of cement, however, would certainly produce the more workable and stronger concrete. Possibly the increased interest in quality concrete has served to overemphasize the importance of elaborate measuring equipment. Simple devices, well supervised, will produce accurate results at a very considerable saving in concrete costs.

In the transportation of concrete from the mixer to the forms, workability is a feature of prime importance. Workable concrete can be transported long distances by various means without appreciable segregation. For short hauls, such as found on building work, the two-wheeled buggy still ranks as the most efficient carrier. Concrete moved in buggies, if very wet, tends to segregate somewhat, all the fine materials rising while the heavy aggregates settle into a solid mass at the bottom. A direct saving in labor in discharging from the buggy results if the concrete is mixed workable enough so that this condition does not exist. Workable concrete when deposited from the buggy leaves the bottom of it clean, and does not require re-working when it is placed in the form.

For longer hauls, where considerable quantities of concrete are to be carried, belt conveyors or chutes are economical means of transportation for workable mixes. In the future, probably more belt conveyor systems will be used than have been seen in the past, owing to their economy of operation and great capacity. It must be kept in mind that neither of these systems will transport a non-workable concrete with any degree of satisfaction. Workable concrete is easily handled, non-workable mixes clog chutes, segregate on belts, and cannot be re-combined at the delivery end into a satisfactory concrete. Conveying concrete by means of chutes has

been unjustly criticized in the past, and equipment blamed in many cases where the real trouble was improperly mixed concrete.

Concrete, containing an excess of water, segregates to such an extent when conveyed by chutes that the heavy aggregate becomes lodged at some point and blocks the chute. Likewise, concrete mixed too stiff is carried very slowly, and is apt to form blockades. The delays resulting while chutes are being cleared are expensive, and point convincingly to the fact that workable and uniform concrete is a necessity for labor efficiency if chutes are used.

In connection with all methods of transportation, the advantage of bottom dumping equipment and its effect on workability should be mentioned. Hoisting buckets, truck bodies, hoppers, derrick skips, and similar equipment tend to remix the concrete, and increase workability if the discharge is from the bottom rather than over the side.

The labor cost of placing concrete in the forms is always increased if the mix is not workable. On many classes of work various consistencies must be used for different portions of the structure. These different parts of the structure may be included in the same day's work, and require an intelligent change in the mix if efficient labor costs are to result. An illustration of this is the case of an inclined concrete slab for a sawtooth roof, which might be concreted monolithic with several bays of flat roof. The concrete for the inclined roof must be placed much stiffer, and be made more workable than the concrete ordinarily required in the flat roof, if the work is to be handled at a minimum cost. In some cases to insure proper placing at low labor cost, it may be desirable to use specially graded aggregates for part of the work, as in the case of fireproofing around the bottom flanges of heavy beams, or in locations where reinforcing is extremely complicated. This means that a knowledge of the workability required for different work must be exercised by those in charge and advantages taken where opportunity permits if concrete is to be placed economically and well. On all structures the surfaces of the freshly placed concrete should be kept as level as possible. Concrete arriving at the forms segregated should be remixed with shovels. The forms should be filled in an orderly manner with few fresh edges left exposed.

Concrete for ordinary structures should be so designed that practically no free water will accumulate on its surface. If pools of water do form as the concrete is placed, they should be removed by shovels and pails. Excess water should not flow over the edge of forms or the face of the concrete, as segregation will result, and the quality of the concrete will be impaired.

On structures which require finished surfaces, workability shows a real saving in labor when the finishing operations are considered. Labor costs for pointing up and repairing honey-combed areas are high and soon over-balance the extra expense that would change a non-workable mix to one of proper workability.

To protect his own interests in the field of concrete today, it is neces-

sary for the contractor to establish some definite method of control and check on the performance of his organization. This may be made quite elaborate and extensive, or it may be comparatively simple as the case seems to require. For ordinary operations, the following system is recommended as a help in obtaining economical workability:

(1) Before work is started, a careful study of the available aggregates should be made and those producing the most workable concrete and giving the highest yield values should be selected.

(2) Proper mix designs should be made and tested for concrete of the strengths and workabilities required by the structure to be built.

(3) Proper facilities for insuring the correct quantities of cement, water, and aggregates should be provided at the mixer. Daily checks on batches of materials entering the mixer and an inspection of the volume of concrete discharged from the mixer are a necessity. At regular intervals, not over two weeks apart, cement and aggregate factors should be computed and recorded, based on total quantities used and concrete yardage placed.

(4) Test cylinders made from different sections of the work as it goes forward, should be broken when one week old and the concrete strength noted in permanent records. A set of these cylinders should be cast at least every other day that concrete is placed. Twenty-eight-day strengths should be computed from actual 7-day tests, and at intervals extra cylinders should be made so that actual 28-day tests can be checked.

By a study of the results obtained in the above manner, the contractor is not only able to assure himself that concrete of the required strength is being placed, but the cost of workability obtained from different aggregates soon becomes apparent. Without some such record of past performance to serve as a basis of comparison in the selection of materials and field methods, true economy in securing workability cannot be realized.

Discussion.—Workability Symposium.

John G. Ahlers (*By Letter*).—A great deal of field work and study has been made on the factors affecting production of good concrete. As a result control and manufacture of concrete is undergoing a slow yet persistent change as we learn more of the effect of each of these. A very important factor is that of workability. The purpose of this discussion is to give information on the relationship between workability and other items that enter into design, control, field production and cost of concrete. Mr. Ahlers.

It must be understood that the term workability of concrete has reference only to that period of manufacture from the end of the mixing operation until final placing in the work. Workability being the last stage has no bearing on other qualities of concrete, but is in turn affected by all of these factors. A greater or lower degree of workability is always obtainable by varying the relationship of constituent materials and thus paid for in an increased or lessened cost of the finished product. Workability of concrete, it must be understood, is apart and distinct from consistency, proportion, strength, design, water-cement ratio, yield and cost. To understand this better the following definitions are offered:

Workability—expresses whether concrete is wet, dry or plastic.

Consistency—that property of concrete which expresses the harshness or smoothness as effected by the particles of stone, sand and cement in relationship to the voids.

Proportions—the quantity combination of: 1, water to cement; 2, fine to coarse aggregate; 3, water-cement to coarse-fine aggregate.

Strength—expresses the resistance of concrete to crushing, shear or tension after it has taken its final set, expressed with reference to the age of the concrete from the time of initial set.

Design of concrete—determination of the proper proportion of water and cement to give the binding medium for the proper proportion of fine and coarse aggregate by trial or careful calculation to give concrete of required strength.

Yield—expresses in terms of quantity or cost the amount of cement, fine and coarse aggregate contained in a finished cubic volume of concrete.

Cost—the total money expended on cement, aggregate and labor in a finished volume of concrete.

The graphic illustration herewith shows how workability is related to all of the above factors. The illustration also shows how workability is a job problem depending on the art of the foreman or laborer controlling the mixing operation, furthermore, that the design of the concrete for strength goes back of and is apart from the problem of workability.

The second diagram, drawn in the same manner as the first, indicates how control of all the constituent materials entering into the manufacture of concrete may be obtained by usual field equipment. By careful and

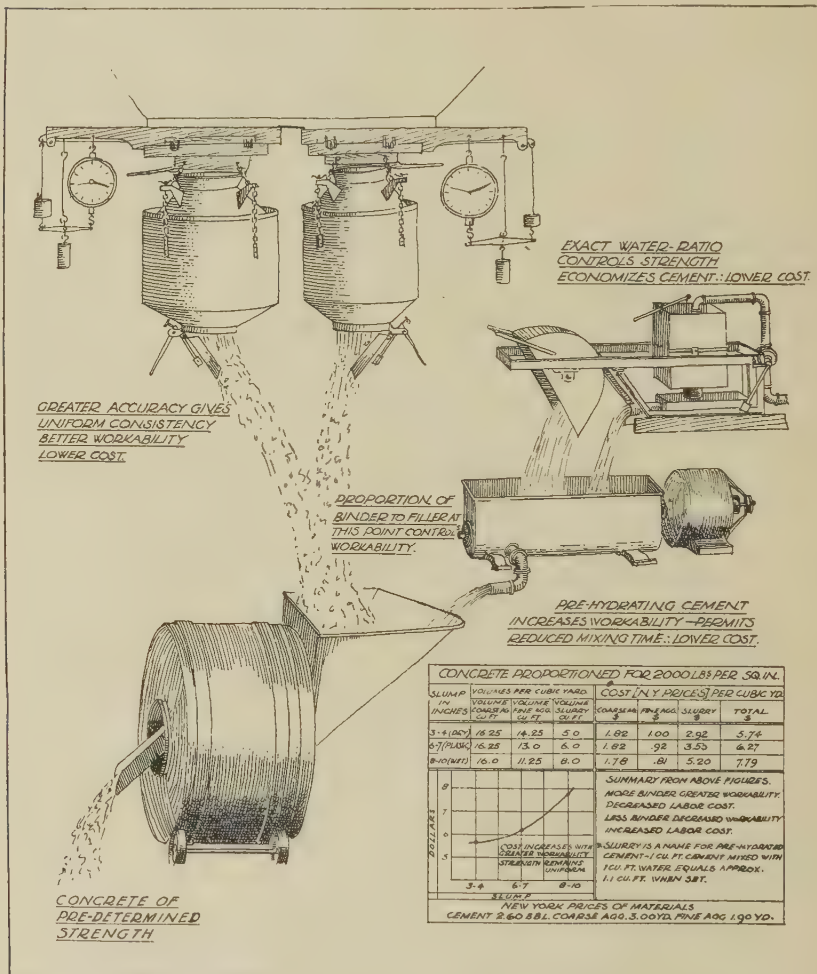

FIG. 1.—HOW CONTROL OF CONCRETING MATERIALS MAY BE OBTAINED BY USUAL FIELD EQUIPMENT.

exact weighing of each separate material the best combination can be worked out to form the artificial stone called concrete. The factor of workability is only an intermediary stage occurring from the time of leaving the

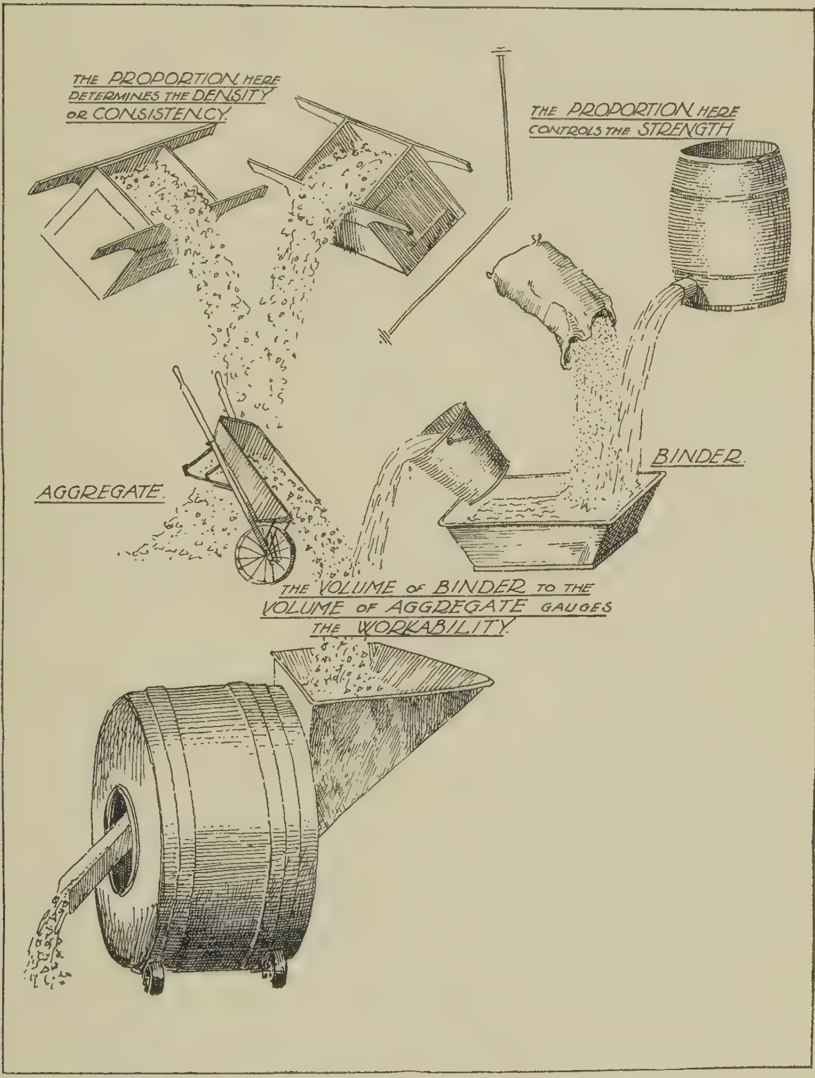

FIG. 2.—RELATION OF WORKABILITY TO FACTORS ENTERING INTO CONCRETE DESIGN.

mixer until finally deposited and placed in the work. This same diagram shows now a change in workability is obtained by changing the relationship between the binder and the filler. The cost of this change is shown in table form.

It will be noted that the manufacture of concrete consists essentially of three parts: first, combining the proper proportions of water and cement, (strength); second, combining the proper proportions of fine and coarse aggregate, (consistency); third, combining the proportions of binder and filler, (workability). In actual practice the three combining actions concur in one mixing operation, but nevertheless the three steps are there. The filling medium, the fine and the coarse aggregate, should be proportioned in relationship to their grading in such a manner as to give the best possible consistency for the finished product. This consistency is often erroneously confused with workability. The consistency and proportions of the aggregates affect the cost of the finished product, because poor grading and proportion require more binder to give plastic workable concrete. The effect on the consistency of concrete from the proportioning of fine and coarse aggregate, and the grading of these, can be seen by trial or by actual calculations.

It is not the purpose of this discussion to set forth the design of concrete for strength or proportion. We assume that the proper water-cement ratio for a desired strength has been determined. Any degree of workability can be obtained then by using more of correctly controlled water-cement paste, or slurry, to a given amount of aggregate. An increased amount of binder will increase the workability of the finished product and also the cost. A better combination of fine and coarse aggregate, or a better grading of either or both, will decrease the necessary amount of binder and therefore increase the workability and decrease the cost. A higher degree of workability can be obtained in the concrete by a longer mixing period. Tests have demonstrated this, and along with a greater time of mixing also is obtained a higher strength of concrete. A similar increase in workability and strength is obtained by prehydration or a thorough mixing of the water and cement prior to being introduced into the mixer. In some test conducted in connection with actual field operations it was very clearly demonstrated that prehydration is most desirable both by observation of the workability and appearance of the concrete, and also by actual test results of test cylinders.

With common control systems a higher or lower degree of workability can be obtained instantaneously by varying the relationship between the water-cement binder and the aggregate. The cost of the resulting concrete will vary accordingly. There is no reason then why in difficult places concrete of a higher degree of workability should not be used without changing the strength, whereas if a lesser degree of workability is usable money should be saved by using less binder for the aggregate.

Remembering Fig. 2 it will be noted that higher degree of workability is obtained at the expense in cost. This cost is worked out from experi-

ence in definite terms of dollars spent for increased slump. But a study of the factors reveals that the same degree of workability or slump can often be obtained by merely changing the grading of aggregates or the proportions of fine to coarse. Prehydration will also increase workability and slump without increased cost of binder or slurry. Therefore study and trial are required to obtain workability without increased cost.

Analysis of the figures herewith will reveal that when admixtures are added they may or may not increase workability, all in the manner by which they require more or less slurry. It is possible to compare the cost of these admixtures with the cost of increased workability obtained by merely using more slurry.

Conclusions.—A—Workability of concrete can be greatly increased by a thorough hydration of the water and cement before adding it in the mixer to the fine and coarse aggregate.

B—Workability can be increased at no cost by better grading and proportioning of aggregate.

C—Workability of concrete can be controlled at will by varying the amount of proportioned water-cement binder to that of the fine and coarse aggregate.

D—Workability can be controlled at the expenditure of a small amount of money for plant and simple mechanical methods.

E—Better workability will decrease labor cost.

F—Using less binder and hence lower workability makes a saving worth while to use more labor in placing.

J. H. WASSON (*By letter*).—For many years it has been standard practice to specify 1 : 2 : 4 concrete for reinforced sections of buildings and other structures. With dry and rodded materials as in the laboratory, this mixture may give a fairly workable concrete. However, with field conditions such a mixture is harsh and poor from the standpoint of workability. **Mr. Wasson.**

For the usual job conditions in reinforced concrete the writer is convinced that if in place of specifying 1 : 2 : 4 concrete the engineer or architect will specify 1 : 2.4 : 3.6 concrete the workability of the concrete will be very much improved, and this without affecting the strength of the concrete adversely. Improved workability decreases honeycombing and adds to the uniformity of the concrete in place.

G. M. WILLIAMS (*By letter*).—The writer would like to see evidence to verify the assumption that ease of penetration of one or more rods into a plastic concrete mass is in any way proportional to the energy required to place the same concrete in actual practice. **Mr. Williams.**

Is the energy required to force a rod into wet concrete, restrained from changing its shape by the rigid walls of the cylinder form, a measure of the work necessary to cause the same concrete to flow into place into a form? Is the energy required to displace and force aside (i. e. to segregate) aggregate particles suspended in a plastic mortar a measure of the mobility of a plastic mass which in practice is unrestrained laterally and flows as a unit without appreciable segregation of mortar from aggregate?

No doubt the results of penetration tests do distinguish and measure some property of the different mixtures being compared, but it is not clear that this property is directly related to or a measure of ease of placing as has been assumed.

The writer would divide "workability" into two divisions: (1) troweling workability and (2) flowing workability. Troweling workability is greatly influenced by the relative quantity of mortar as well as the plasticity or wetness of the mortar. Comparing two mixtures having the same flowability but differing in relative mortar contents, the one of highest mortar content would generally be the most easily trowled and finished. The mixture containing the greatest volume of mortar could be surface finished with the least expenditure of energy. Troweling workability is especially desirable in mixtures used for the surfacing of floors and in concretes used for paving where a trowel or float is employed in the finishing process. The writer believes that the penetration apparatus is of value in measuring this property.

Flowing workability or flowability is a necessary quality of concretes used in mass or structural work where the mixture must have a certain fluidity or mobility in order that large masses may be transported and placed homogeneously with a relatively small expenditure of energy. The plastic, semi-fluid concrete mass is caused to flow into final position, thoroughly filling the form and surrounding the reinforcing steel, by shoveling or spading of the concrete and vibrating the forms and the reinforcing rods. Where there is considerable reinforcing steel present, ease or difficulty of placing is dependent directly upon the flowability of the concrete. The more readily flowable the concrete may be, the smaller the expenditure of energy needed, as measured by vibration of the reinforcing steel or form. The spreading of a mass of concrete on the flow table under the action of a given amount of energy is comparable to the ease or difficulty of placing the same concrete in the forms. For all plastic mixtures free from segregation, the writer believes the spread of a mass of concrete or mortar on the flow table is a measure not only of consistency as defined by the authors but also a measure of the workability or placeability of the concrete for structural purposes. Tendency of a mix to segregate is evidenced on the flow table by the rolling of coarse aggregate particles beyond the mortar boundary, if the mass is too dry or too harsh, or by the spread of the mortar ring beyond the body of the aggregate if the mass is too wet. Either condition represents a less workable mix than one of the same measured flow which shows no tendency to segregate. The flow table when properly used is not only a measure of flowability but of the tendency for any given concrete to segregate. After nine years of use of the flow table the writer believes that relative flowability as well as consistency of structural concrete is measured by the flow table since the action of the flow table is comparable to the methods used for transporting and placing concrete in its final position in the work.

The results obtained with the old and new types of penetration appara-

tus by Messrs. Smith and Conahey are interesting in showing how resistance to penetration decreases as the mortar content is increased or altered by the addition of various powdered materials. If flowability is maintained constant more mixing water must be added to those mixtures containing powdered additions and the effect is generally to slightly lower compressive strength and increase the absorption and permeability of the concretes. In this connection it is interesting to note that marked differences in penetration can be obtained by using sands of different granular analyses or fineness. Pearson[1] and Hitchcock found that whereas the workability figure for the sand (called "normal sand") throughout a series of tests was 65, the use of a finer sand gave a workability figure of 18, superior to that obtained when any integral powdered material was used with the normal sand. Furthermore, there was no loss in strength when the fine sand was used, while without exception all powders which increased "workability" tended to lower compressive strength. The writer would suggest that the use of a greater relative proportion of the "normal" sand would have tended to decrease the workability figure in the same manner as the use of the finer sand, although perhaps not to the same extent. So long as concrete mixtures of the required flowability show no tendency to segregate there would seem to be no advantage in altering the mortar content or its condition. When segregation tends to occur its bad effects may be overcome by: (1) the use of a powdered admixture; (2) the use of a finer sand; or (3) the use of a greater relative proportion of the original sand. Sands which have low surface areas due to lack of fine particles are especially troublesome when high flowabilities are required and one of the above methods should be used to reduce segregation. In any particular case the most practical and economical method can best be determined by actual tests.

Whatever the distinction may be between workability and consistency the important and necessary quality in the placing of a structural concrete is the "flowing" quality, and the writer believes that the flow table furnishes an accurate measure of this quality.

JOHN G. AHLERS.—Just one word to amplify my written discussion. **Mr. Ahlers.** I propose to stick to one single point, the question of slurry, or the water cement mixture, which, after reading the papers, seems to be the outstanding point that governs the workability of concrete. By using more or less of that water-cement paste, which controls the strength, you can get any degree of workability with the materials you have in hand. Mr. Doe said in his paper that this is the thing that interests people who make concrete and have to do it at a reasonable cost in order to make it feasible to do the work at all. The discussion submitted was intended to focus on this phase of workability; what does it cost to get workability? There was, for instance, a table which shows that concrete with an increase in slump from 4 to 8 in. would cost two dollars more per cu. yd., using the same

[1] A. S. T. M. *Proceedings*, 1923, Vol. 23—Part II.

material and the same water-cement paste. In looking at that phase of the subject I believe there is a possibility for getting additional workability without increasing the cost by trying out in the field what has been done in the laboratory by Nathan Johnson and some of the mixer companies, that is, pre-hydrating the cement in the water. From some actual field work I am fully convinced that if a practical way can be found for pre-hydrating the cement, you can increase your workability equivalent to the addition of slurry and save at least $1 per cu. yd.; that is the point that the discussion was intended to focus thought upon. The question of workability depends on the slurry which controls the fluidity and thereby controls the cost and strength of the concrete.

Mr. Douglass. A. S. DOUGLASS.—I would like to take a minute on another ingredient of concrete, another element of workability which seems to me has not been mentioned. You have heard the college professor's joke in which he told his students that there was a fourth ingredient of concrete of which they had not thought, and that it was water. I could tell you of another ingredient of workability concerning which the program committee, in lining up this group of papers, has not thought of. Twist the word "workability" and say "ability to work." I think that is probably the greatest ingredient in good concrete for the contractor and for the engineer. On a small piece of road pavement being done with a ½-yd. portable mixer as part of a larger job and by a concern carefully watching and proportioning its concrete, the inspector, who was in full authority as to the preparation and design of the mix, had turned out the first batch from the mixer. The general foreman, a typical old timer, complained about the unworkability of the mix. The inspector was figuring on the back of an envelope to determine how much more water and how much more cement should be added, when the general superintendent appeared and said, "What is going on?" The inspector said, "The general foreman has asked for a softer mix." The superintendent said, "Wait a minute. If you give it to him, how much cement are you going to add per batch?" He told him. "About how many batches are you running per hour?" He told him. Then turning to the general foreman, he said, "How many men will you save if he gives you that softer mix?" The foreman told him, and the superintendent, after a little thought, said, "Get busy and place it as it is, and thereby save, two to one, the price of that extra labor." Understand, I do not suggest that that can be applied to highly reinforced walls. It can, however, be applied to a great deal of work. I want to call attention to one statement in Mr. Doe's paper which apparently might be taken as insignificant but which to me is highly important. He said, "When surplus water occurs on top of concrete, it should be removed by shovels or buckets." I am firmly of the opinion that on first-class, controlled concrete, workable enough for any class of work, no water should appear on the top, and I am speaking as a user and placer of concrete and not as a person having anything to sell.

I also agree with the statement in Mr. Ahlers' paper, which says, "Using less binder and hence lower workability makes the saving worth while to use more labor in placing."

C. M. CHAPMAN.—I have been particularly interested in the paper on methods of measuring workability. The difficulty with the contractor, the concrete placer, is to distinguish fine differences between mixes as to their workability. He must depend for his information regarding the workability of his output upon the opinion of the workmen who are placing it, the opinion of the man who looks at it as it comes out of the mixer or the opinion of some inspector. Until we have a method of determining with some accuracy this quality which we call workability, we are going to make progress slowly in utilizing its advantages. The paper by Mr. Smith and Mr. Conahey is very interesting, but I do not see in it evidence that what they are measuring is the workability that the contractor is talking about. Until we know that what they measure or give a numerical value for is the quality which enables the workman to put that concrete into place with less labor and at a lower cost, the usefulness of the data obtained with this apparatus is limited. We cannot apply it until we know they apply. I wonder if the authors of that paper can point out whether that quality which they measure is the same workability which the contractor is looking for? Mr. Chapman.

GEORGE CONAHEY.—If you will refer to Fig. 8 of the preprinted paper you will find the results of a series of tests made to see if the penetration apparatus would measure the property of concrete known as workability. You will find the work done in driving the rods 11 in. into the concrete plotted along ordinate and the "flow" of the concrete plotted along the abscissa of Fig. 8. The concrete with flow of 40 on the left hand side of the diagram was very dry and almost crumbly. The concrete with a flow of 150 on the right hand side of the diagram was extremely wet. There are very few things about workability that concrete investigators will agree upon. Most of them, however, will agree that a moderately wet concrete is more workable than a dry concrete when both are made of the same aggregate and cement. Most of them will also agree that when two concretes are made of the same aggregates and cement and are mixed to the same relative consistency or have the same "flow" the concrete containing the larger proportion of cement (richer mix) is the more workable. Mr. Conahey.

If you will examine the curves of Fig. 8 you will find that more in. lb. of work were required to drive the rods into the dry concrete (flow 40) than to drive them into the moderately wet concrete (flow 100). It also required more work to drive the rods into the concretes containing the smaller proportions of cement when mixed to the same relative consistency or "flow." The work done in in. lb. in driving the rods 11 in. into the concrete is called "the workability figure" of that concrete.

The mixes shown, 1:1½:3, 1:2:4, 1:2½:5, and 1:3:6 were made of the same gradation of the same aggregate and the same brand of cement. The only difference between these concrete mixes was the proportion of cement to aggregate.

Water was added to the concrete to give 5 different relative consistencies as measured on the flow table for each mix. In obtaining these data a batch was prepared and tested at a certain relative consistency, then it was discarded. Three tests were made on each of 5 batches to give the data for each point on the diagram. If you will examine the cures you will find that the results line up in the manner you expect a workability apparatus to show them. Take the 1:1½:3 curve for example. At the point showing the dry concrete (40 flow) it requires a relatively large amount of work to drive the rods into the concrete, or the concrete has a high workability figure. This dry concrete is more difficult to place than a wetter concrete of the same proportions. When the concrete was mixed to have a flow of about 70 the results of the tests with the penetration apparatus show this marked difference in the workability, that is the workability figure was considerably lower at this consistency (70 flow) than at the dry consistency (40 flow). The results of the tests with the penetration apparatus also line up in the order expected when more water was added to cause a longer flow, i. e., with a flow greater than about 80 relatively large additional quantities of mixing water have very little effect upon the workability of a 1:1½:3 concrete.

The effects of increased quantities of mixing water on the other mixes is similar to its effects upon the 1:1½:3 concrete, except that it causes segregation in the lean mixes and the concrete is more difficult to place in order to obtain a uniform finished product. The workability curve goes back up showing this segregation.

The segregation is most noticeable in the lean (1:2½:5 and 1:3:6) mixes. The mixing water does not show as great an effect on the workability of the lean mixes as on the workability of the rich mixes.

The penetration apparatus also measures the difference in the workability of the concrete due to a change in the proportions of the cement. A 1:2:4 concrete mixed to have a flow of 100 is more difficult to place than a 1:1½:3 concrete. It also has a higher "workability figure" than the 1: 1½: 3 concrete. Similar comparisons can be made of the other mixes.

In making the tests on these different mixes the water cement ratios of the concrete were varied over a wide range. If the water cement ratio instead of the "flow" were plotted along the abscissa of Fig. 8, the curves would not line up one above the other as they are in this figure. They would be separated in the horizontal direction as well as in the vertical direction. The curve of the 1: 1½: 3 mix would occupy the extreme left hand portion of the diagram, the 1: 2: 4 curve would be a little farther to the right and above it, and the 1: 2½: 5 and 1: 3: 6 curves would be still farther to the right and still higher above the others.

It was because these results were parallel to the way the workability of a concrete is estimated in the field that we considered that this property which the penetration apparatus measures, was the workability of the concrete.

F. R. McMILLAN.—In reference to the last remark that Mr. Conahey made in regard to Fig. 8 of his paper, there is one factor that is ignored that will considerably modify the conclusions which he has drawn. There is no question at issue in the selection of concrete for a particular job that could be answered by reference to this series of curves. Before he can answer the question that Mr. Chapman has raised, it will be necessary for him to present data as to the relative workability of a number of different mixes of the same strength. All of our concrete jobs are predicated on some definite quality, strength or something of that kind, and there is no use comparing a 1 : 1½ : 3 mix of 6 gal. of water per sack of cement as to workability with a 1 : 3 : 6 mix of 10 gal. of water per sack of cement; they cannot be considered on the same job, will not perform the same service, and there is no occasion for comparing them on that basis. The same may be said to a certain extent in regard to Fig. 12 in which he compares a number of mixes discussed in Table 2, as to relative workability, without saying anything as to the suitability of those mixes for a particular job. Until we see the data comparing these various mixes which are equally usable on our jobs, we are at a loss to decide as to the merits of these tests. Mr. McMillan.

GEORGE CONAHEY.—There are a number of things that must be taken into consideration. When we first started this work we had hoped to find that the concrete users were not discussing altogether whether their concrete would be 3,000, 4,000 or only 2,000 lb. per sq. in. at 28 days. We wanted to know something about the ease of placing concrete and we have been devoting all our efforts to that end. We have used the 1 : 1½ : 3, 1 : 2 : 4, 1 : 1½ : 5 and 1 : 3 : 6 mixes simply because we knew there would be a definite difference between these mixes, and we felt that if we used the same gradations, of aggregates, and the same cement, we would get mixes with which we could try out different methods of measuring workability. Mr. Conahey.

Now the contractors are interested. If they can obtain the strength that they want with a 1 : 3 : 6 concrete but find it costs too much to place such a mixture, they want to know what they can do to improve the workability. They want to study the effect of the addition of more cement, a change in the gradation of aggregates or perhaps the addition of admixtures which was mentioned by Mr. Doe. Our paper has been limited to laboratory work. Thus far we have not been able to line up our laboratory apparatus with the work in the field. We hope to do that at a future date, so that we can compare the results that we get with our laboratory apparatus with workability as it is estimated in the field.

A. B. MORRILL.—Referring to Fig. 12 and Table 2, it seems to me that the only fair way to compare these concretes is to compare concrete of equal value. Just below Table 2, the authors state "Pearson and Hitchcock found that the workability of a concrete mixture was about equally improved by the addition of 9 lb. of hydrated lime, 6 lb. of kaolin or 3 lb. of celite per bag of cement or by the use of 25 lb. of additional cement." Mr. Morrill.

Table 2 shows a considerable difference in the water-cement ratios of the samples tested, a much greater difference than should be allowed on a carefully controlled experiment. The water-cement ratio with celite added is 1.119 and the water-cement ratio for plain concrete is 1.005. This difference alone is sufficient to cause a great difference in workability aside from the addition of any admixtures. Further, the authors have added 25 per cent more cement with no increase in the amount of water, decreasing the water-cement ratio to 0.86 and obtaining a very superior concrete. This last concrete must have been very much better than any of the others. It would seem only fair when adding more cement to add more water in the same proportion, keeping the water-cement ratio constant; if that were done the workability would have been obtained with much less than 25 per cent more cement.

Mr. Conahey

GEORGE CONAHEY.—The different concretes were tested as nearly as possible at the same relative consistency as measured by the flow table. To obtain the same relative consistency it was necessary to vary the water content of each batch. Additional water was added to the batch containing the 25 lb. additional cement. If you will notice the two asterisks behind the water cement ratio 0.864 you will find that it refers to a note which reads "based on all cement used." In other words, enough water was added to this batch so that it would have a flow of 90 to compare with the other mixtures.

There is another point which must be considered in judging these results. If you will study the effect of admixtures on the water-cement ratio strength relation of a concrete you will find according to the data of Abrams and also according to the data of Pearson and Hitchcock that a higher water cement ratio can be used with the concretes containing admixtures than can be used with the concretes without admixtures, and the same strengths obtained.

Mr. Pearson.

J. C. PEARSON.—Mr. Conahey has referred kindly to the work of Mr. Hitchcock and myself in the search for a method of measuring workability as something different from slump. To raise the question at this time of what effect on concrete strength any measures which may be taken to improve workability may have, is only to becloud the issue and divert attention from the purpose of the investigation. Primarily Mr. Conahey's object is to develop a method of measuring workability. If we subscribe to the desirability of having such a method, then why bring up the subject of quality—that is an entirely different matter. Like Mr. Chapman, I think the method is the important thing right now, because until we can measure workability, we cannot discuss these other questions as to whether water-cement ratio or admixtures or anything else will yield concrete of definite value.

In regard to Mr. Conahey's methods, I still have a lingering doubt in my mind as to whether they will prove adequate, because either the penetrometer or the squeezometer, according to the descriptions given, seems to register its indications in rather arbitrary units. It seems to me

that we may have to get back to the more fundamental flow-pressure relations, such as Mr. Bates has described in his studies of cement. Of course we cannot squeeze 1½ aggregate through a capillary tube, and apply that method to concrete. In concrete we shall have to deal with the properties of shear and force, or more specifically with the rate of shear, presumably through a stirring device of some sort, and the power required to drive the stirrer. I have always felt that this problem was going to be most hopefully attacked by some sort of a concrete-mixer device in which the power applied to the paddles would be analogous to pressure and the speed of the paddles to the rate of shear or flow. We cannot tell beforehand whether the revolution of the paddles in the mixture will give true shearing values or not, but the scheme appears promising, if only by its resemblance or analogy to the methods commonly used in studies of plastic materials. Mr. Pearson.

But this is only an opinion; I have not had opportunity to try the method suggested, and can only express the hope that someone will try it. What I wish to register here is appreciation of the work of the authors of this paper, and hope that they and others will continue to search for simple and dependable methods of measuring the plasticity or workability of cement mixtures.

P. H. BATES.—I am afraid we have made a little god out of strength and felt that it was the main thing, whereas it is only a measure of the final thing we are after—serviceability. We can obtain serviceability to a far greater extent in a great many structures by workability rather than by strength, and therefore the main question to be considered is workability. Mr. Bates.

A. S. DOUGLASS.—I would like to state some facts. My position is such that I am responsible both for quality and cost. The contractors have not that enviable middle ground to view the problem from. We discovered from extensive tests that we could get consistently—and the figures are all subject to irrefutable proof backed by voluminous records—reliable, continuously, 2000-lb. concrete with so little cement that I felt it necessary, in order to get enough cement into concrete calling for lower strength, arbitrarily to dictate that no concrete should be prepared with less than 5 bags of cement. For conditions where extreme protection of steel is not entirely necessary we are now using a minimum of 5 bags of cement for ultimate 28-day, 2500-lb. concrete. With about 6 bags, we are designing 3000-lb. concrete and using that for our reinforced work. In other words, on our cheapest concrete we have increased the customary stresses 25 per cent and on our high-grade concrete we have increased our working stresses 50 per cent. Mr. Douglass.

E. M. BRICKETT.—Mr. Conahey's paper deals with laboratory tests in which it is absolutely necessary to have a common basis for your measurements. Mr. Conahey selected the basis of equal flow or equal slump. Now certain men here of a practical trend are rather confusing the issue in stating that he should have adopted equal stress. Mr. Conahey is attempt- Mr. Brickett.

ing to measure the workability of concrete, and that is the first thing we must have and it is the thing toward which this discussion should tend.

Mr. McMillan. F. R. McMILLAN.—I would not want the opinion to go out that I am not in agreement with much of the comment here on the value of this work. My previous remark had only to do with the question of whether this was a measure of workability. If Mr. Bates or Mr. Pearson are in doubt as to my stand on the importance of workability, let them read some of the questions and answers in the Primer which will be presented at a later session. I think I go as far as either of them on the question of workability as an essential to the durability and the placement of concrete. My point is that it is unfair to ask us to rate this particular test as a measure of workability until it is tried out on the mixes which are otherwise suitable. It is a very excellent test of the penetration of concrete, the same as the slump test is of the slump of concrete and the flow test of the flow of concrete, but does that mean that it is a measure of the thing we want on the job? It may be. It is probably better than our slump or flow test, but before we are asked to accept it, let us see it tried out on mixes that are equally suitable for the work.

Prof. Talbot. A. N. TALBOT.—I should like to ask a question concerning the meaning of Fig. 8, to be able to judge of this relation between the new instrument and flow table work. I think nothing has been brought out as to how the variation in each of these mixes, say the 1:1½:3, was obtained. I judge from what has been said that it was through an increase in the amount of the mixing water. That being so, will Mr. Conahey be good enough to tell us where the consistencies in ordinary use would come. There are two points in going from the small flow to the higher flow that give an inclination; the others show about the same amount of workability factor with a considerable increase when the flow is derived from the flow table. What is his explanation of that?

Mr. Conahey. GEORGE CONAHEY.—The concrete with a flow of 40 corresponds approximately to the concretes used in road construction. The concrete with a flow of 100 corresponds to the concrete ordinarily used in reinforced-concrete building construction. The flow of 150 is a soupy concrete which is often used in thin wall sections. The man who is building a large dam of mass concrete is interested in a concrete of an entirely different workability factor than the man who is building a concrete road or reinforced-concrete building. As a rule a moderately wet concrete does not segregate. If the concrete is placed too dry it will give trouble with honeycombed spots. The concrete must be wet enough to be put in place and a compact mass formed with the equipment available on the job, if good results are to be obtained.

If you will refer to Fig. 8 of the paper you will see that the 1: 3: 6 concrete with a flow of about 40 had a lower "workability figure" than did the same concrete with a flow of about 60. This was due to the fact that the concrete with the flow of 40 was not wet enough to be properly compacted by the method we used in preparing the test specimen. When

more water was added and the flow of 60 obtained, a dense uniform sample was the result, and it required more work to drive the rods into this specimen than into the other. All our efforts have been expended in trying to develop a method of measuring workability. Until we feel confident that the method will measure the differences in workability of the laboratory mixes there is no need of testing concretes made on the job, for our efforts will be wasted. We believe that the penetration apparatus does measure the property of concrete known as workability, and we hope to be able to make tests with concrete in the field in the near future.

A. N. TALBOT.—I judge then that this instrument shows little difference in workability whether the flow is 80 or 120 or 150, and also—I am giving the figures named by Mr. Conahey—whether the slump is 3 in. or 8 in. or 10 in.? **Prof. Talbot.**

GEORGE CONAHEY.—This data was obtained with a one-rod apparatus. Substituting three rods makes the apparatus more sensitive and we will get a great difference between 1 : 1½ : 3 concrete and 1 : 2 : 4 concrete. **Mr. Conahey.**

ALFRED H. WHITE.—This symposium has discussed cement and water and aggregates and Mr. Douglass added labor. I would like to ask about another factor, the time of mixing. **Mr. White.**

GEORGE CONAHEY.—In making up these specimens we followed what I think is the usual laboratory practice; we proportioned our mixes by volume, but we weighed the materials. They were placed in a shallow mixing pan and mixed dry until they were of uniform color. The water was added and the mixing continued for 2 minutes. In determining one of these points we made 3 tests on an individual batch and made up 5 batches, so that each one of the points on Fig. 8 is an average of 15 tests. Due to the nature of the apparatus, a large number of tests must be taken into consideration. We recognized this, and we tried to get in as much of the data as possible without going into such detail that no one would read the paper. When we considered the effect of the time of mixing, considering the elapsed time between the 3 tests as mixing time, we found that the average of the last tests was lower, or that the concrete was more easily worked than when the first test was made. We hope to study this problem further at an early date, but at the present time we have not been able to obtain sufficient data to state just what would be the effect of a longer time of mixing. **Mr. Conahey.**

ALFRED H. WHITE.—I understand that these 3 points were made on the same mix. One test was made, then after a lapse of 3 or 4 min. another test was made and then, after 3 or 4 min. more, a third test, so that the first test was after 4 min., the next after 8 min. and the next after 12 min. on the same batch and the plasticity increased? **Mr. White.**

GEORGE CONAHEY.—Yes, sir; the plasticity increased. **Mr. Conahey.**

ALFRED H. WHITE.—Suppose in concrete work the cement mixer was run 5 min. instead of 1 min., what effect would that have on the workability of the mix? **Mr. White.**

GEORGE CONAHEY.—That would increase the workability, but we have **Mr. Conahey.**

not been able to develop an apparatus that is sensitive enough to tell us to what degree. That is another reason why we are using 3 rods instead of 1 in the penetration test. We are trying to get a more sensitive apparatus. It is generally admitted, I think, that increased mixing time will increase the workability of the concrete. Just how much it will increase it, we hope we can determine. If any one has any ideas we will be glad to hear about them.

Mr. Wiepking. C. A. WIEPKING.—It is clear to me now that this test was made entirely for developing apparatus and to investigate apparatus for use in testing workability. It is in laboratory process. It will be up to some one to continue this work and apply it to the field. There is no attempt being made to say that any of these values or any of these mixes can be applied directly from these charts to any of our mixes in the field. There is no data regarding the character of the sand, and for that reason we cannot very well apply these values numerically to any field work. The proportion of fines in the sand is known to have an effect on the workability, and especially in cases of mixes like 1 : 2½ : 4 which are on the border of being under sanded. For that reason, in an illustration like Fig. 12, the data shows simply how this particular apparatus measures the workability or flowability, and does not say anything about the economy, the strength or the durability of any of these concretes under test.

Mr. Spurney. F. E. SPURNEY.—From the contractor's point of view we are interested in amount of work necessary to turn over a mass of concrete. Mr. Conahey described four methods of measuring workability. Are there any more? As Mr. Doe asked, "Is it going to take more electric power on the motor to turn over a workable mass than it is to turn over a stiff unworkable mass?" With such an idea in mind, we made a miniature mixer drum, filled it half full of concrete and rolled it down an inclined plane. We thought at first the internal friction of that mass, i. e., the distance the drum rolled out on a level plane, would be a measure of the workability. We did not get very conclusive results, and we put a corrugation on the inside of the drum. We did not get any conclusive results with that, either. We are now wondering what would be the result if we put blades in the drum?

Mr. Brown. H. W. BROWN.—There are one or two things about Fig. 8 that have not been brought out. What we want to know is whether this horizontal line, say 200, represents a very workable concrete such as would be required in the most difficult reinforced-concrete work; whether 400 workability concrete can be used in ordinary reinforced-concrete construction, whether we can run up to 800 for road work, and what should be the upper and lower limits. If we can show by field tests with this apparatus that these lower horizontal lines are really horizontal, we have a very valuable measure of workability. It might well be that in the future we will say road concrete must have a workability based on tests with this apparatus.

Mr. Munsell. A. W. MUNSELL.—I do not get the idea of pressing of the rods through

the concrete, which is not a practical way of measuring the workability of concrete. In depositing concrete around reinforcement, the concrete passes through the reinforcement and because of lack of workability the large aggregates hang to the bars, particularly where crushed stone is used the aggregates arch and form air or stone pockets. This may be due to the lack of some lubricant, in some cases, a too lean mortar or under-sanding.

My idea in making a test for workability would be to design an apparatus similar to actual conditions. In other words to pass concrete through several planes of reinforcement and measure what comes through. I have in mind an adaptation of the Jackson apparatus described in the *Proceedings* of the A. S. T. M. several years ago, in which is used an inverted cone-shaped vessel with a sliding door at the bottom, to hold a certain amount of mixed concrete. Below the inverted cone is a spring balance with a table one foot square. The amount of concrete held on the table is the measure of consistency.

I propose to use an inverted cone-shaped vessel in which reinforcement bars in two planes, spaced 3 in. centers horizontal and vertical, would be fixed, the openings in the two planes of bars to be staggered. In operation, the vessel would be filled with concrete and struck off. The sliding door would be pulled and the amount of concrete held on the table of the scale would be the measure of workability.

The bars through which the concrete would pass would give that interference met with in actual placing of concrete and would more nearly simulate actual conditions than that proposed by Mr. Conahey. No work has been done on this apparatus yet, although we expect to start soon.

A. N. TALBOT.—Some of the speakers have suggested forms of apparatus similar to those used in various tests made at the University of Illinois 5 to 10 years ago in a series of efforts to develop a test that would measure the resistance to the internal movement of the particles of the concrete among themselves (the movement of the particles relative to one another), which if it could be measured would be the real test of workability as it is needed in placing concrete in construction work. One method, tried in different ways, was to stir the concrete with revolving arms or paddles and to measure the current taken by the electric motor that actuated the apparatus. In many of the forms of apparatus tried, a large part of the current or the effort required to actuate the apparatus is taken up by resistance other than that involved in workability, so that with many variables in the resistance of the concrete to motion the indications accompanying changes in the quality of the concrete are small and indefinite; and these troubles are likely to be inherent in many of the methods proposed here today. **Prof. Talbott.**

Another difficulty that will be encountered lies in devising a method that will be suitable for mortars and for a range of mixtures of concrete with aggregates of different size and gradation. A device that seemed promising in advance consisted of a rubber tube (an inner automobile tire

tube was tried) connected with a short cylinder at one end. When this was filled with concrete and a weight was applied to a piston that moved in the cylinder it was expected that the weight required would be proportional to the resistance to movement of the particles of concrete among themselves as the tube expanded under the pressure. Actually the pressure in the tube was not uniform over its section, a central core carrying the main part of the load when the tube expanded as does ballast under a railroad tie. In all forms tried there was some obstacle in the way of success. It is to be hoped that efforts to devise an effective method of measuring workability will be continued. It is a most important problem and some day a solution will be found.

Mr. Drusbach.

E. E. DRUSBACH.—In the mixtures as indicated in Fig. 12, did Mr. Conahey find the same amount of water necessary in these mixes to get the necessary flow?

Mr. Conahey.

GEORGE CONAHEY.—You will see the water-cement ratio expressed in Table 2. In answer to a question as to whether it is worth changing the mixing time from 1 min. to 1¼ min., this depends on just what you want in workability. I know a products man who changed his mixing time from 1 min. to about 3 min. and was able to increase his output from 10 or 12 blocks per bag of cement to 16 to 18 blocks per bag. If you are mixing trowel concrete, the effect of changing the mixing time from 1 min. to 1¼ min. is not very great. We have no method of measuring workability, and until we do we will have to use our own judgment.

Mr. Upson.

PRESIDENT UPSON.—Mr. Conahey, I think it is probably wise for you to amplify your statement by saying that your final strengths were not greater for the long mix. The products manufacturer gained not from an increase in strength but from an early strength. We might be misled on that.

Decorative Painting on Concrete.

By Sidney F. Ross.*

In 1926 our firm made its initial study of decorative painting on concrete. This was described in an article written for the *Architectural Record* and appeared in the April, 1927, issue. For that reason it is hardly necessary to present the matter in detail, but for those who are not familiar with what we tried to accomplish a brief outline may be of interest.

We had originally contemplated using a wood ceiling for the new Cadet Mess Hall at West Point, New York, but on account of the size and excessive cost of this work other treatments were considered, and it was finally decided to try the experiment of painting directly on the rough concrete surface of the flat slab and beam construction. With that idea in mind the ceiling was designed with relation to the beam spacing, false concrete beams being added to carry out the design as required.

In the meantime, we had started a Temple House for the Union Temple of Brooklyn, which contained an auditorium also having a beam ceiling. As this building was in advance of the construction of the Cadet Mess Hall, we initiated the scheme for decorating painting on the concrete ceiling of this auditorium. The construction used was cinder concrete, and care was taken that the form work should be a better-appearing job than is usually demanded. It was found, however, that this proved to be immaterial, as the grain markings and joints were almost obliterated by the under paint coatings. The texture, which was in some respects rough and uneven, gave a certain quality that would not have been obtained on a plastered surface.

Numerous experiments were conducted using a number of brands of cement paints and coatings to fill the pores of the concrete and prevent efflorescence and discoloration through the finished painting. It happened that the building had to be finished in a very short time and the work was therefore started while the concrete was still green. For that reason the problem was an unusually serious one and offered many difficulties that had not been anticipated. It was soon found that the various paints used were not effective and discoloration appeared even after two or three coats had been applied.

At this point we consulted Nathan C. Johnson, and through his efforts developed a formula for the under coating which has proved entirely efficient. It appears that concrete as a material is totally unlike any other

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substance known. The data on the chemical reactions extend back for a period of perhaps sixty years, during which time it has been found that concrete is subject to constant change somewhat similar to the breaking down and renewal of the body tissues. Only a small percentage of the cement used in the mixture is taken up when the concrete has its initial set, and as time goes on more and more of this unused cement becomes active. Another factor to be noted is that the bottom of the form is covered with a rich skin cement coating, which dries faster than the remaining mass concrete, and in so doing a tremendous tension is exerted which shortly causes fractures extending in many directions throughout the under surface. Therefore, the base coating for the decoration, to be permanent, had to have considerable penetration, and had to be of a substance not affected by dampness or by the chemical reactions in the concrete causing deterioration and disfiguration or permitting efflorescence and saponification. It was also necessary to present a surface capable of receiving and retaining oil paints and pigments without destroying their permanency.

The formula finally devised contained a cellulose base, to which was added a certain amount of color to permit visual inspection, insuring a thorough coating of all surfaces. After this base coating had been applied it was found that an actual penetration of three-eighths to one-half of an inch had been attained. The nature of the material used was similar to many of the lacquer preparations and had an extremely disagreeable pungent and sickening odor; for that reason and also to obtain the maximum amount of penetration it was planned to spray the original coating. In this, however, difficulties were encountred, as the Unions would not permit painters to operate a spraying machine, and we were unable to obtain a favorable ruling. It was, therefore, brushed on by hand, the painters using masks, but even with this protection they were unable to work for longer periods than one-half hour at a time, and a similar period had to be allowed for the atmosphere to clear before work could be resumed.

As an added precaution a second base coat was used consisting of a specially prepared lead and oil paint with the addition of various pigments. The decorative painting was applied over this. As the ceiling has been completed for over a year and shows no sign of discoloration whatever, it is safe to assume that the method employed was successful.

The ceiling of the main dining room of the Cadet Mess Hall at West Point is an extremely large one, covering an area of approximately 70 x 500 feet. This will be decorated with the same materials used for the Temple House. As the construction, however, is of stone concrete instead of cinder, the under film of cement is unusually hard, and it is anticipated that the amount of penetration will be considerably less. For that reason a solution of aluminum was painted on the forms, which exploded on coming in contact with the wet concrete and broke up the cement coating in small globular-shaped openings of various sizes which will give an additional key of adherence for the base paint coatings.

There seems to be no reason why painting on concrete is not entirely practical for certain types of decoration, and it offers a much more pleasing appearance than a smooth plastered finish or one artificially applied. In addition to that there is, or should be, a material saving in the cost over a furred ceiling. There is also a saving because of the omission of the bond coating of plaster where the paint is applied directly on the concrete. The one previous drawback has, of course, been the almost certain discoloration, but by the use of simple precautionary measures this can be overcome.

Discussion.—Painting on Concrete.

Mr. Johnson. N. C. Johnson.—There were two very interesting problems, and I think they are of a good deal of interest to the concrete industry. The first one mentioned, the cinder concrete ceiling, was a particularly bad ceiling; efflorescence was extremely heavy and the water saturation was excessive. The second problem was one of stone concrete and to prevent scaling of applied paint, an effort was made to put perhaps billions of tiny pores into the concrete at such a period that they would remain as pores. I am very glad to say that this effort was entirely successful.

Cellulose acetate paint and primer was used throughout. Cellulose acetate is non-soponifiable and has many advantages over the usual oil-vehicle paint.

I did not know until this paper was read that in the first case mentioned the body paint would be a cellulose acetate paint, and had assumed that merely the primer alone would be a cellulose solution. But some advanced means of painting was necessary because we had a very poor ceiling, exceedingly porous and requiring a masking of all its chemical elements.

The second case was a different problem, consisting, first of all, in producing a porosity such as would allow the paint or primer to go up into the concrete so it would not scale off.

The chemical side of the question has been touched upon. The subject of effective painting on concrete is intricate and may not be gone into thoroughly here. But we might say that it is not merely a matter of climate and humidity in relation to concrete as to whether or not one paint or the vehicle for a paint will be successful, one as against another. In Los Angeles they are painting very successfully, I understand, directly on the concrete without any intermediary, but the climate there is absurdly stable as to humidity. In a very dry climate, likewise, you may do many things that you may not do in a moist or humid climate, and most of all, in a climate of varying humidity.

Reinforced Concrete as Applied to Monumental Buildings.

By Emil Praeger.*

The word "applied" seems to be out of place in a discussion of modern concrete construction. "Adopted" would probably be more in keeping with the remarks that I have in mind. This is a very broad subject and might easily include the majority of all reinforced-concrete structures, the word monumental as used in this instance meaning "conspicuous and lasting."

In the east there have been a number of conspicuous structures executed in reinforced concrete such as athletic stadia, arch bridges, etc. There have also been a great many conspicuous reinforced-concrete buildings constructed in this part of the country, but for the most part these structures have been limited to industrial types of buildings with little attempt or necessity to improve the usual set scheme.

You may have noticed that I used the word conspicuous and not lasting. An article recently appeared in one of the New York papers, evidently following Sir Edwin Luyten's statements concerning the lack of permanence of American buildings, which stated that there was only one building in New York City which was being built to last 100 years or more and that was St. John's Cathedral, which is being built almost entirely of granite. I agree with the assumption that the granite structure should stand at least 100 years, but I also feel that the author of this article has been entirely too exclusive.

Reinforced concrete is a comparatively new building material and it has made wonderful strides in the past 15 or 20 years. The possibilities with this material seem almost unlimited, but at the same time it now has its limitations. The work which such organizations as the American Concrete Institute, the Portland Cement Association and the various research departments connected with engineering schools throughout the country are performing will greatly aid in lifting some of these existing limitations. We seem to meet fewer of those reinforced-concrete specialists who try to convince you that reinforced concrete is the best material for any and all purposes. I am not a specialist in that sense of the word, because I must admit that there are places where structural steel, stone, terra cotta, wood or other building materials can be used to better advantage than concrete.

Recently there seems to have been an effort to create a characteristic concrete style of architecture. This is an effort which is commendable, as

* Engineer, Bertram G. Goodhue Associates—Mayers, Murray & Phillip, Architects, New York City.

there has been altogether too much sham connected with recent architecture, especially in some commercial buildings with structural steel frames. While I am not at all in favor of sham in building construction, I do feel that the continued development of reinforced concrete has made it possible to depart from the traditional forms where necessary and make use of shapes and elements not possible with any other material. At the same time we are at liberty to use reinforced concrete in many of the places where only stone, wood, brick or other materials formerly could be used. If desired, the same style or type of ornament may be used with concrete as would be the case with stone or with wood. We have often heard that no ornament should be used unless it has a useful structural purpose. In my opinion this is an altogether unnecessary restriction and one which need not be considered too seriously. A building devoid of all ornament would certainly be an uninteresting structure.

Before concrete was available, the architect was restricted within narrow limits for his selection of shapes and sizes of structural members. With advance in knowledge of the possibilities of this material the architect will be obliged to modify his ambitions less often than before, and the more capable that we become as engineers the fewer will be the restrictions placed upon the architect.

While reinforced concrete offers unlimited opportunities for the designer's imagination, he should not throw tradition to the wind, forget everything that has been accomplished in the past and try to create a new architecture overnight to fit this new material. Let us not become slaves to concrete, but rather develop new forms slowly and as the requirements at hand seem to demand.

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As stated, there have been comparatively few reinforced-concrete buildings erected in the east other than those of an industrial type. Let us therefore turn to southern California, where the use of concrete has been more varied. Some 4 or 5 years ago, Mr. Bertram Goodhue was commissioned architect for the new Central Branch of the Los Angeles Public Library, the appropriation for this work being about $1,500,000. The building covers an area approximately 200 x 300 ft., or 60,000 sq. ft., on a considerably larger plot of ground, the main wings being 3 stories high with a central tower 135 ft. above the main floor level. For the architectural design as well as structural, economical and other considerations seemed to make the use of reinforced concrete ideal.

In Fig. 1 are renderings of the first two studies of the Hope St. entrance, together with the finally adopted plan. In the first study the windows had circular heads with engaged wall columns. In the second study these details were somewhat simplified by using flat-headed openings and rectangular pilasters. These changes made a more economical concrete construction and at the same time improved the design. In the second study the dome and central tower have been made higher, but this scheme

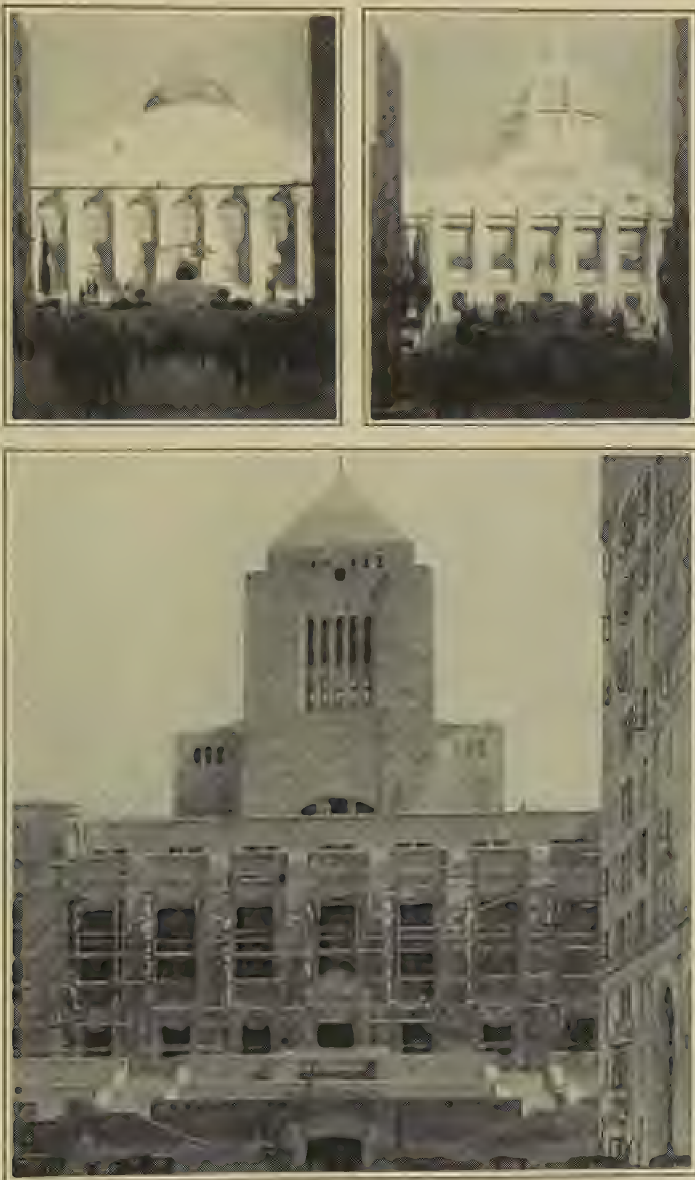

FIG. 1.—PRELIMINARY ARCHITECTURAL STUDIES AND CONSTRUCTION VIEW, LOS ANGELES PUBLIC LIBRARY.

was abandoned in favor of the square tower and peaked roof as shown. With this final scheme the tower has been utilized as a stack space, with very little increase in cubage over the second scheme and probably no extra cost.

As may be noted in Figs. 1 and 2, the exterior consists of concrete wall columns, concrete spandrel beams and concrete curtain walls. All of this concrete has been hidden either with stucco or limestone trim. While we have no serious objection to the use of stucco or stone, there is here an opportunity for those interested in advancing reinforced concrete, namely, the improvement of the poured concrete so that it will first not need sur-

FIG. 2.—COMPLETED WEST ELEVATION, LOS ANGELES LIBRARY.

face treatment, and, second, that it will not be necessary to use stone for decoration and ornament.

True, there have been many buildings constructed which have exposed exterior concrete walls, but while the texture may be quite satisfactory, the natural color of the poured concrete may be dull and uninteresting if not at times actually ugly. Then again the concrete may be subject to crazing and temperature cracks, it may be honeycombed in spots and it is difficult satisfactorily to patch even minor blemishes. An architect cannot afford to gamble with these conditions. Obtaining a concrete of sufficient strength and one repellent to water and other elements is of course extremely important, but this phase of the subject will not be dwelt upon, because the secret of obtaining these qualities is known even though they are not always attained.

In regard to the substitution of concrete for stone ornament, this can only be accomplished by constant study and practice by architects and contractors. Ornament, figures, etc., should be poured with the building structure to effect the greatest economy and latitude of design. Obtaining a concrete of a more pleasing color and one that will be weather-resistant without additional treatment also will undoubtedly help to solve this problem.

It may here be noted that in designing this building the probability of earthquake shocks had to be considered. Here again the advantages of

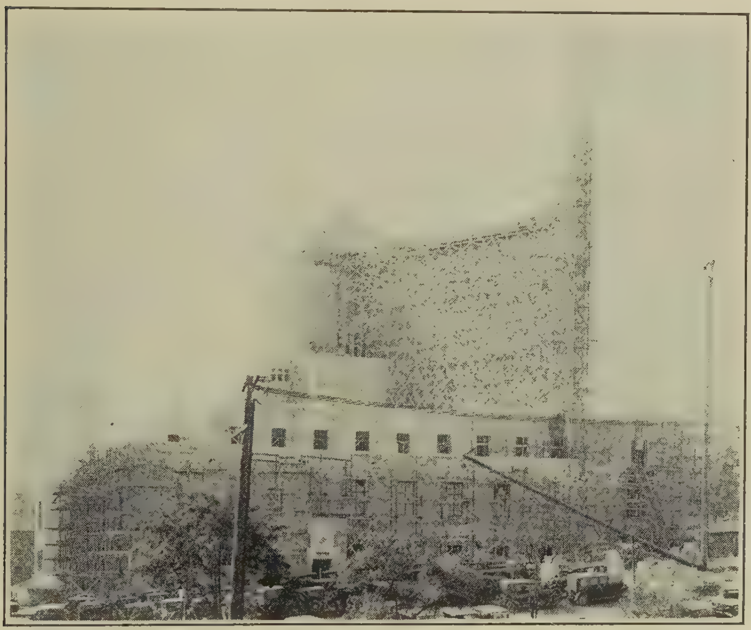

FIG. 3.—LOS ANGELES LIBRARY UNDER CONSTRUCTION.

reinforced concrete are apparent. Recently the practice has been to design buildings for a seismic factor of 1/10, that is, it is assumed that a severe earthquake would produce a horizontal force on each floor equal to 1/10 the dead weight of that floor. This force can be transmitted through the floor system to the wall or other rigid bents and distributed among these bents according to their rigidities so that only a small proportion need be resisted by the interior flexible bents which are made up of the interior columns and beams or girders. Earthquake forces, while never occurring at the same time as wind, act in a similar manner and are from 2 to 5 or more times the magnitude of wind forces. Well-built concrete buildings

have given a good account of themselves in past earthquake disasters, but, nevertheless, continued study on this subject is advisable.

In regard to the interior treatment, in almost every room of importance the structural members were featured rather than hidden, and the results seem to have been very satisfactory. Floor beams and girders were located so as to be symmetrical about the axis of the room below, or in such other positions as were desirable from the designer's viewpoint. In all cases, the architect and the engineer worked close together. The ceilings of practically all rooms were left exposed, the concrete beams and girders being decorated with stencil or other painted decorations.

FIG. 4.—READING ROOM, LOS ANGELES LIBRARY.

Note that structural concrete members are featured, not hidden, and that they are decorated with stencil.

The ceiling of the rotunda is a reinforced-concrete dome 38 ft. in diameter, which acts as the support for 4 tiers of library stacks. This dome is supported by 4 reinforced-concrete arches which also carry the walls and roof of the tower. These surfaces were decorated without attempting to hide the board marks or otherwise disguise the material. Furring and lathing and plaster were eliminated wherever possible.

The cubical content is approximately 3,500,000 ft. and the cu ft.-cost, including plumbing, heating, elevators and mechanical equipment, was about 38¢. There undoubtedly are many less satisfactory buildings today which have cost three or four times this amount. In spite of the

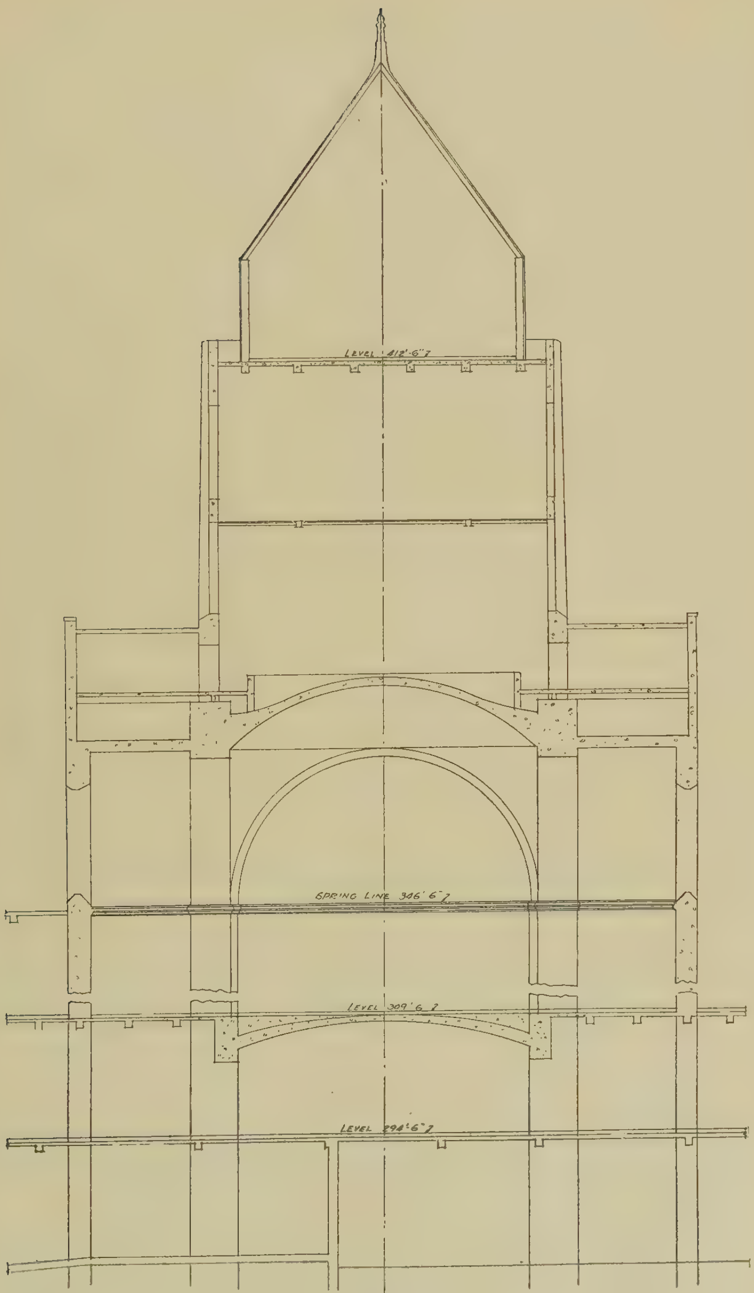

FIG. 5.—SECTION THROUGH CONCRETE FRAMEWORK, LOS ANGELES LIBRARY.

fact that the owners' requirements were considerably increased during the progress of our working drawings, the cost of the building (approximately $1,300,000) was less than 90 per cent of the original appropriation and probably less than 75 per cent of what the client expected it would be. Therefore, in spite of the fact that very elaborate architectural and structural detail drawings were necessary (the building department of Los Angeles requires drawings giving complete details of every member including actual bends of rods and also lists of bending moments, shears, etc., before approving same) the architect had the pleasant experience of foregoing his expected commission on the amount which his design saved for the client. Here, then, is another phase in which there is room for improvement. I say this quite seriously, because the suggested improvements or variations noted previously, namely, omission of exterior stone and stucco and interior lathing and plastering have the effect of considerably reducing the cost of the building without necessarily sacrificing its utility or beauty. At the same time these changes increase rather than decrease the amount of detail work which the architect must perform, and with each saving effected the architect further penalizes himself by the reduction of his commission. If concrete is to be used for monumental types of buildings it must attract the best architects in the country, and to do this it seems that a more equitable basis of fees should be developed than the percentage of cost basis now generally in use.

Church of the Heavenly Rest.

Our office was recently commissioned to design a place of worship for the congregations of the Church of the Heavenly Rest and the Chapel of the Beloved Disciples. The site selected was purchased from an owner whose home occupies half the entire city block to the north on Fifth Ave. The property was purchased on condition that the ridge of the roof of the church be not higher than 85 ft. above curb level. One of the conditions imposed by the client was that every pew should have an unobstructed view of the altar.

The usual type of construction consists of a vaulted ceiling over which there is a steep light roof supported by steel roof trusses. The restriction of height made this type of construction impossible, since an 80-ft. ceiling height was necessary from a design standpoint, leaving only some 3 or 4 ft. for construction at the center line. We, therefore, first decided to use a structural tile vaulted roof of a construction similar to the tile ceiling.

However, when a careful analysis of the stresses with this type of construction was made it was found that a stone buttress some 7 or 8 ft. greater than the width available was necessary. After some trials we became convinced that a satisfactory solution could be obtained by using reinforced concrete. In place of the outer tile roof a reinforced-concrete roof was substituted using a poured nailing concrete slab to receive the lead-coated copper roofing. This slab is supported by reinforced-concrete beams which in turn are supported by the reinforced-concrete arch ribs.

The ribs are part of the reinforced-concrete piers which extend to the rock foundation.

In this particular building the concrete construction must serve its purpose without praise or glory because it is to be entirely encased. On the exterior, the walls are of Indiana limestone, while in the interior the walls are of Tammany buff sand stone and the ceiling of acoustical tile.

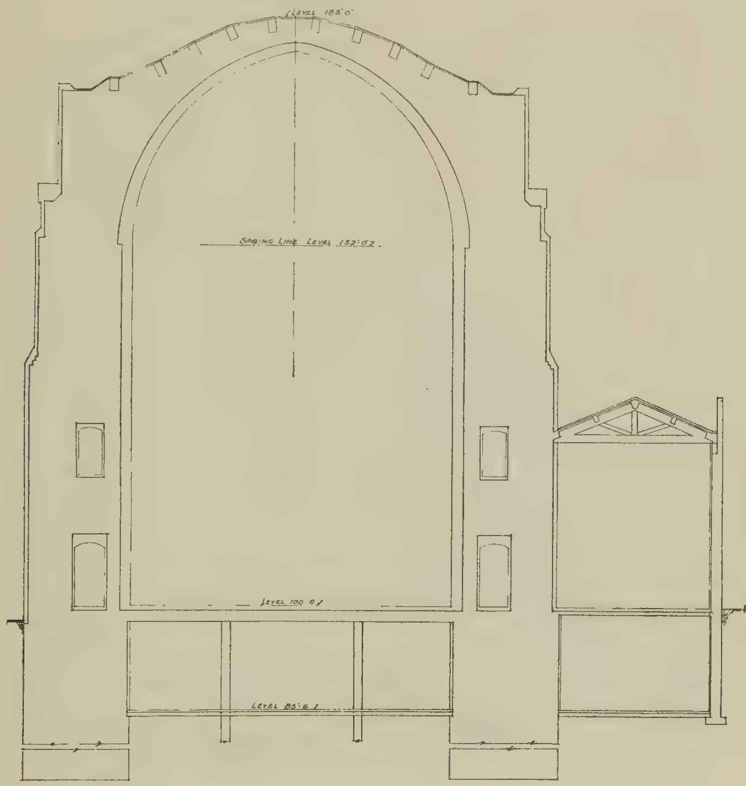

FIG. 6.—CROSS SECTION, CHURCH OF HEAVENLY REST, NEW YORK CITY.

There were several possible methods of designing the arch ribs, but it was decided, after many trials, to consider each of these ribs and piers as two cantilever bent beams touching at the center. With this assumption the only bending moment at the crown is that caused by temperature changes and secondary stresses caused by possible deflection. From the crown to the buttress the bending moment is negative (that is, tension at the top) and increases gradually to a maximum at approximately El. 162,

where it begins to diminish until it changes sign at about El. 152, before the vaulted ceiling is in place and at El. 134, after the vaulted ceiling is in place. Below this level the sign is positive, that is, the bending moment produces tension on the interior and compression on the exterior side of the pier. It was decided to start the reinforcing directly under the nave floor so that at this location the bending moment was zero. With this

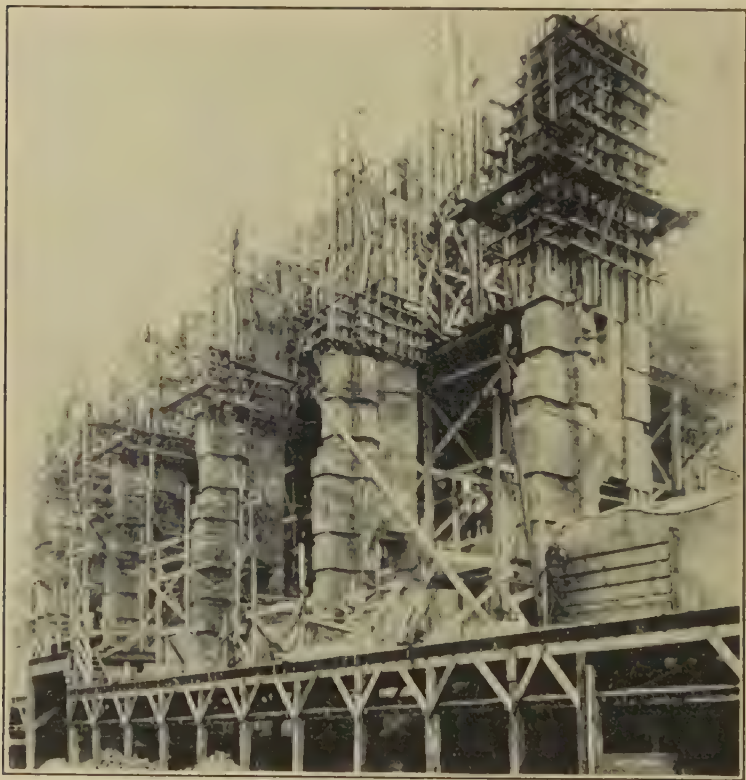

FIG. 7.—PIERS OF CHURCH OF HEAVENLY REST SHOWING RECESSES FOR BONDING FACE STONE.

known factor, the bending moments at all other sections could be determined and the stress in the concrete and reinforcing computed.

There are several constructional features which might be mentioned. The stone work is of ashlar coursing, not less than 4 in. thick, while every fifth stone is a bond stone 8 in. thick. Our specifications required the general contractor to submit drawings showing the exact locations of all recesses in the concrete to receive the bond courses as indicated on the

stone contractor's setting drawings. In order to expedite the work, however, we prepared drawings giving the exact location of all stone joints and the necessary locations of slots and recesses to receive the stone. In addition, dovetail anchor slots and galvanized anchors were specified and installed to tie the stone back to the concrete. One inch was allowed between the back of the stone and the face of the concrete for grout. At junctions

FIG. 8.—CORBELS AND SEATS FOR ARCH RIBS AND VAULTED CEILING, CHURCH OF THE HEAVENLY REST.

of concrete piers and brick walls the concrete was toothed so that the brick courses bonded exactly with the concrete; wall anchors were also used. It was of course necessary to provide skew-backs at the springs of all arches, ribs, vaults, etc., and in addition, corbels were provided at different levels so that the weight of the spandrel wall was taken by the concrete piers, thus tieing the entire structure together. This is shown in one of the construction photographs.

Although the value of the stone was not considered in the structural design, tied and bonded to the concrete piers as it actually is, it unquestionably adds considerable strength to these members, and can be considered as an additional safety factor. Stone work was not started until after the concrete had been poured for several weeks during which time the concrete was kept moist (especially in warm weather) by allowing water to trickle over the top of the section last poured.

There have been a number of stone-faced buildings in which the stone has cracked quite badly. This may have been caused by unequal settle-

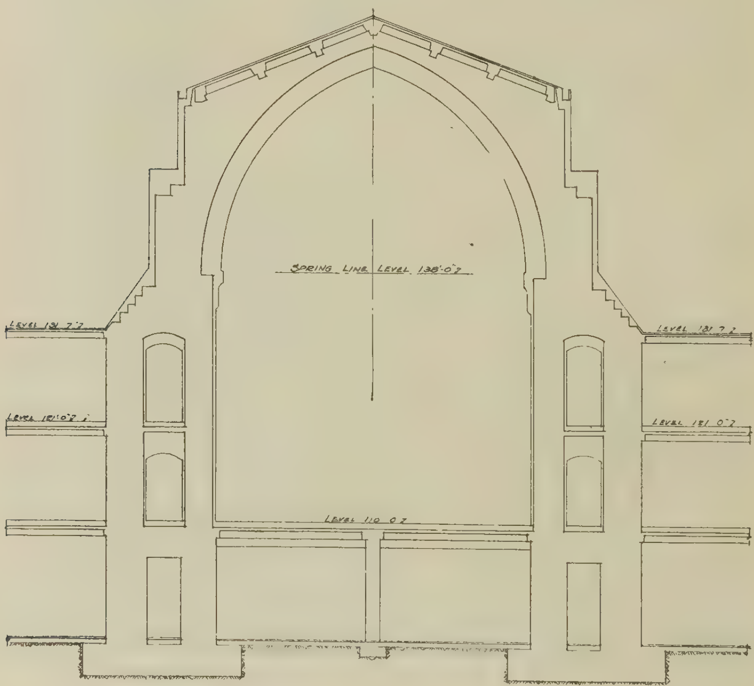

FIG. 9.—CONCRETE FRAME, EUCLID METHODIST CHURCH, CLEVELAND.

ment of the stone due to shrinkage of the mortar joints, different rates of expansion and contraction of facing and backing with temperature changes, or other similar causes. This movement might load some stones beyond their carrying capacity and therefore cause them to crack. To overcome this condition, a lead shield in which a corrugated lead sheet is sandwiched was placed in the alternate bond courses on the top of the bond stone. The thickness of this lead joint is ¼ in. and as the stone joints are ⅜ in. we were able to put a thin levelling bed of mortar above and below the pad. If there is any unequal settlement the movement will not act over a

greater height than the distance between pressure relieving joints, and there is more than enough elasticity in this joint for it to act as a spring and to prevent excessive load coming on any one stone. These shields are kept back about ⅜ in. from the face so that we can point the joint with elastic cement or with cement mortar which is not tooled in tightly.

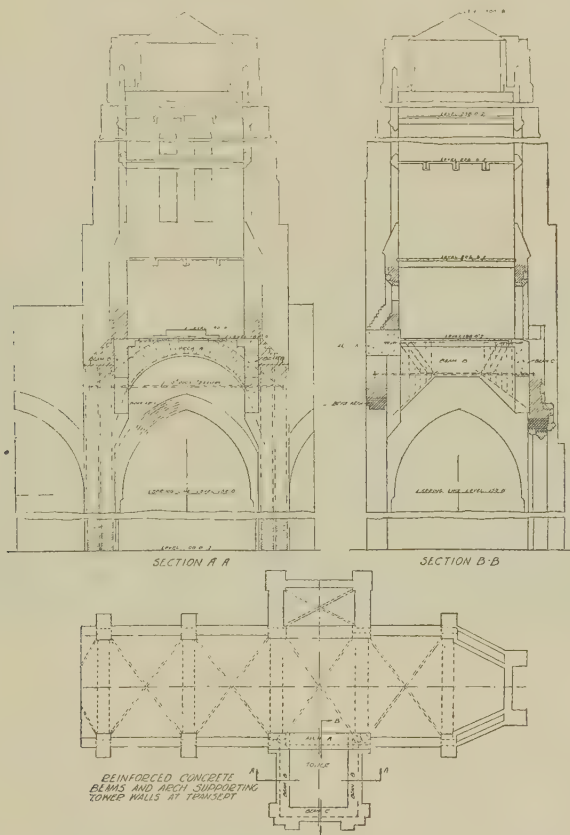

FIG. 10.—CROSS SECTION, CHICAGO UNIVERSITY CHAPEL.

Note that exterior faces of upper part of tower are inside of inner face of lower part of tower.

In another instance on this job reinforced concrete played a rescue roll. Heavy timber trusses were designed for the roof of the chapel, but the fire department objected, and a very satisfactory reinforced-concrete truss was substituted. In this case the concrete roof beams and concrete trusses are exposed and are to be decorated.

The stone mason advises that he has experimented little or no trouble in setting the facing. The concrete piers and arches present an imposing appearance and with the development of a better and more uniform concrete it should be possible, when considered desirable, to build such a structure without the use of stone. While the finish treatment of this church does not necessarily indicate the structural design and materials, it is in my opinion perfectly honest construction, in contrast to the somewhat

FIG. 11.—UNIVERSITY OF CHICAGO CHAPEL.
Tower supported by concrete arches as shown in Fig. 10.

usual scheme of forming large interior piers of plaster imitating stone with a small steel column within.

OTHER MONUMENTAL CONCRETE BUILDINGS.

In a church which is now nearing completion in Cleveland a somewhat similar structural design to that of the Heavenly Rest was followed. Here, however, the concrete roof slabs and beams were exposed and are to be painted without further treatment. In this case the structural roof and floors of the adjacent sunday school rooms simplified the problem of resisting the arch thrusts.

We have also resorted to similar types of construction in other churches. For example, we used such concrete construction in a church in Fort Wayne, Indiana, in springing 4 arches, which supported the inner

walls of a crossing tower, from the sides of 4 other arches which paralleled the first set but sprung from a lower elevation.

In the Chicago University Chapel the wall of a side tower at the crossing was supported by a reinforced-concrete arch, the spring of which is 75 ft. above the floor level; the walls of the tower rise an additional 120 ft. above this spring level. Here the thrust was taken by heavy tie rods. This problem was complicated by the fact that the tower walls weathered back so that the exterior faces of the walls above the roof of the nave were inside of the interior wall surface below (Figs. 10 and 11).

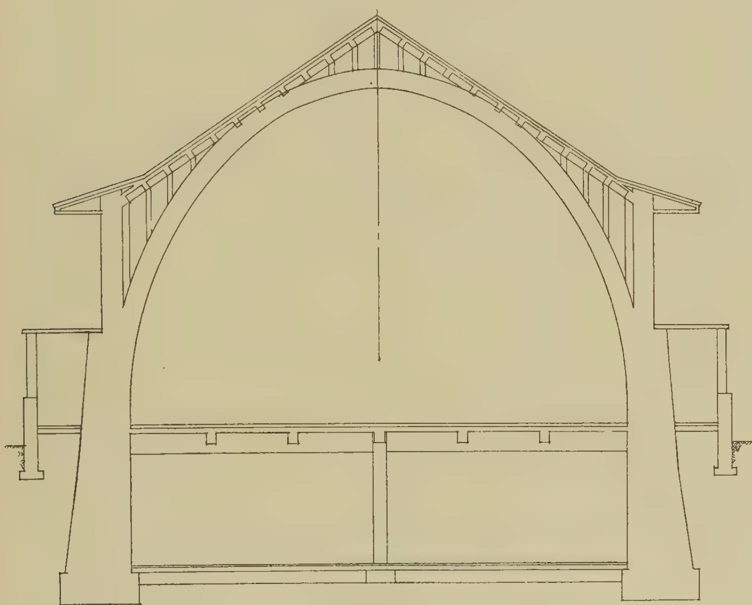

FIG. 12.—DILLINGHAM HALL, HONOLULU.
Concrete used in place of wood because of danger from termites.

To take care of this condition at the three sides, deep reinforced-concrete beams intersecting each other at the supports were used. Reinforced-concrete columns faced with stone supported these members.

The Church of the Redeemer, now under construction in Brooklyn, N. Y., has an octagonal tower with pointed openings at the ceiling below. As there was no opportunity to provide sufficient buttress for these arches, the walls were designed as concrete beams, thus eliminating the necessity of providing such buttresses. This, with concrete, is perfectly honest construction.

About a year ago we completed the first wing of a monolithic-concrete school building at Glenbrook, Conn., in which exterior structural walls

were colored by the admixture of about 2 per cent by weight of cement of a mineral yellow. This gave a pleasing buff color which required no further treatment. Wide overhanging eaves on this building will probably be an aid in protecting the wall surface from the effects of the elements. I do not know of any other buildings in which the color of the poured concrete was similarly predetermined. Board marks should not be obliterated since, when properly placed, they add considerably to the texture of the exposed surfaces.

Within the past few years we have designed and had constructed some eight or nine buildings at the California Institute of Technology. Some of these have been very special in their character and requirements, but there has been little difficulty in adapting reinforced concrete to their construction. Provision for earthquake stresses has been one of the most important requirements.

The building for the Bank of Hawaii, at Honolulu, is almost entirely of reinforced concrete, but the front and side elevations are faced with cast stone. A deep coffered ceiling was used in the banking room. The structural members in this ceiling as well as practically all other ceilings were decorated by stenciling the poured concrete members. Concrete also was advantageously used in the construction of the heavy money vaults.

In Dillingham Hall, Honolulu, we had originally intended to form the roof of laminated wood arches. There is a species of bug in Honolulu (termite) which is very destructive to wood, and at the request of the client these arches were revised slightly and designed for reinforced concrete.

I would like to repeat that in my opinion there is a great future for reinforced concrete, but at the same time there is room for improvement, especially with regard to the treatment of concrete which is to be exposed to the weather under varying conditions. I would earnestly suggest that this and the use of color in concrete be given the maximum amount of study.

Discussion.—Monumental Buildings.

N. C. Johnson.—Our good friend, Harvey Corbett, who started out as an engineer and has excelled as an architect, has defined the engineer as "an architect with all the juice squeezed out of him." **Mr. Johnson.**

I could not help thinking of this mordant characterization as I listened to this excellent paper, because I believe that the cause and art of reinforced concrete will advance further and advance more rapidly if engineers will forget stresses for a while and think objectively as to the net result to be achieved as does the architect. This gentleman who has spoken for himself and his associates is also speaking for a multitude of architects in this country and abroad who are thinking all the time in objectives.

I trust, therefore, it is not an impertinent paraphrase of this paper if I say that in summation of this utilization of concrete by an architect, we find that "the ceiling is of tile, all exterior exposed surfaces are of stone, all interior exposed surfaces are of plaster or stone or brick or tile and the sub-foundations undoubtedly are of concrete."

That is a paraphrase, of course, but it is all that the concrete world at large deserves, for concrete has been used, thought of, investigated and researched solely on the idea of a stress-bearing material. By contrast, we are accustomed to seeing objectives obtained in architecture by brick. We may build a brick wall without lateral support or reinforcement to twenty times the basic width. But because so much weight has been attached to strength as the sole attribute of concrete, we may not, under the stupidity of our codes, build a concrete wall even to three times the base thickness without putting so much steel in that we almost cut it in two.

And the cost is correspondingly high.

Yet a brick wall is nothing but a concrete wall with a special aggregate; but if we take that same brick, run it through a crusher, mix it with some mortar and make it in the same form, we must pay a heavy penalty in reinforcement.

One reason, then, that architects may not and do not extensively use concrete except for buried work is the code. A concrete wall is worth many times a brick wall in lateral stability, in strength and in every other virtue, but under the code we must consider it an inferior material and put in so much steel, which must be mild or intermediate steel of low elastic limit, that we are required to build such an enormous box to hold that steel that architectural expression by line and by form is an impossibility. No wonder that the architect throws up his hands and says, "What's the use? You cannot express anything in concrete except a crude, pre-medieval conception of what a structure should be."

So I hope that engineers will forget strength for a little and begin to study the architects' needs a little more thoroughly.

It is the greatest inspiration in the world to get away from thinking about 28-day strengths. None of us ever built anything to endure just 28 days, and we are not horribly interested in what that strength is, so if you will pardon this remark, I would say to those who ostensibly have progress in concrete as a heart's interest, "Forget your old prejudices. Forget your hard and fast ideas. Forget your assumed virtues and begin to really investigate the material that you talk about so freely, but do not begin to know. And above all, ask the architect what he wants in concrete beyond footings and basements. Then find ways to give him what he wants."

If engineers will do that for two years, this Institute may have a new vision of the value of Concrete in Architecture; and architects and engineers alike, a new revelation on Architecture in Concrete.

Reinforced-Concrete Walls for Buildings.

By W. E. Hart.*

The purpose of this paper is to develop discussion on the use of reinforced-concrete filler or panel walls for buildings and to point out some of the economies to be gained through the use of this construction in place of masonry walls. The construction consists of erecting a reinforced-concrete building of any height and enclosing the building as it is erected with reinforced-concrete walls. The forms for these walls are built in such a manner as to include practically all of the architectural details and trim. When the forms are stripped a monolith stands practically complete except for interior finish. The walls for such a building may be treated in several ways in order to produce the desired color effects.

Introduction.—As a basis for the structural design of buildings as discussed in this paper, let us conceive the simplest type of fireproof construction. The factory or loft building probably would give the greatest floor area at the lowest possible cost per square foot. The walls, columns and floor systems would be made of reinforced concrete placed by common labor with intelligent supervision. Rough holes would be left in the walls for windows, doors and other openings. Such a construction would represent the maximum structure at a minimum cost for materials and labor. From this point the building may be given any treatment desired in order that it may be suitable for the class of occupancy for which it was built. Such a construction can well serve as a foundation from which to design office buildings, hotels and apartments. In fact, this construction might well be used on fully 90 per cent of our present building programs.

In such a building concrete may serve as a structural and architectural material. This concrete must be designed with care and the construction supervised by competent men. The concrete for this building is to be exposed to the action of the elements and for that reason must receive unusual care and attention in making and placing. The contractor should bid the concrete at a price that will permit the care and attention required by the architect. In other words, the concrete for this type of building must be recognized as a different concrete than that used on the normal building job, because in this case it has the additional function of finishing and decorating the building. Further on definite recommendations are made as to the mixes that will produce a proper service in the concrete. Suffice it to say here that the old-time 1 : 2 : 4 concrete does not

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possess the characteristics necessary to resist exposure or produce architectural embellishment.

Buildings using exterior walls of monolithic concrete have been used for factories and industrial developments in the central and eastern part of the United States with satisfactory results so far as housing the industry is concerned but with exceedingly poor results from the architectural

FIG. 1.—BAPTIST CHURCH, LOS ANGELES.

This church illustrates what may be done in an untreated concrete. Five and one-half-inch form lumber produced an excellent exterior. All ornamentation, except the Rose Window, was cast in place.—Allison and Allison, Architects.

viewpoint. It has fallen to the lot of the architects and engineers on the Pacific Coast to demonstrate the practicability of using concrete successfully as a structural and architectural material. All of the examples used in this paper are taken from the Pacific Coast with particular reference to the advancement being made in southern California. A survey of buildings from Seattle to San Diego will convince the most skeptical mind that

concrete can be used successfully as an architectural building material. England, France, Germany and many of the South American countries have used concrete as a complete building material for many years while our own west coast brings this construction home to us.

The Present Situation.—Before taking up the details of the problem let us look at present-day conditions with reference to what is involved

FIG. 2.—TOWER OF SEARS-ROEBUCK & CO., LOS ANGELES.

The straight, vertical lines of this tower adds height and dignity to the building. The ornamentation at the top of the tower was cast roughly in the forms, and afterward run truely with a cement plaster.—George C. Nimmons & Co., Architects.

and why certain methods have been used. Since 1919 building permits have been on a steady increase, each year surpassing those of the previous year. In 1927 there was a slight drop which in the minds of most statisticians is only a hesitancy rather than the start of a general decline. During these progressive construction years, the demands for new building materials and new assemblies of building materials have steadily increased to keep pace with construction. A premium has been placed upon labor

so that today we pay from 100 to 300 per cent more for the same class of craftsmen over that paid in 1914. As a rule a large percentage of the cost of a building material is represented by the cost of the labor required to produce that material. Therefore, economic conditions have placed a certain restraint upon the manner in which buildings shall be erected. In face of all this, the architects of America have fostered and built more than thirty-five billion dollars worth of excellent structures in the past six years.

FIG. 3.—SEARS-ROEBUCK & CO., LOS ANGELES.

This structure is a monolith having exterior walls, columns and floor systems made of the same concrete and placed at the same time. The exterior treatment is of an ivory portland cement stucco.—George C. Nimmons & Co., Architects.

Compare the architectural design of office buildings today with those built in 1900. The façade is simple, with straight lines and a minimum architectural treatment between the third and fourth floor belt course and the sky line. This is due in a great measure to the cost of carving and setting ornamentation. In other words, the architect has been forced for economic reasons to reduce his architectural embellishment to a minimum.

The fact that economics must be practiced in present-day construction does not relieve the situation from the architectural standpoint. In other words, the architect still has the desire to design buildings with an embellishment suitable for the structure. The fact that concrete can be produced, where a duplication in ornament is permitted, more economically than any other form of treatment makes this new material desirable from

the architectural standpoint. It costs as much to make the first model in concrete as it does to carve the same model in stone. If the model is produced twice the original cost is cut by one-third. After four or five duplications have been made, the cost is so greatly reduced that this form of treatment does not become a serious item of cost.

The question has often been asked by members of this Institute, "Why do not architects use more concrete on the outside of their buildings?"

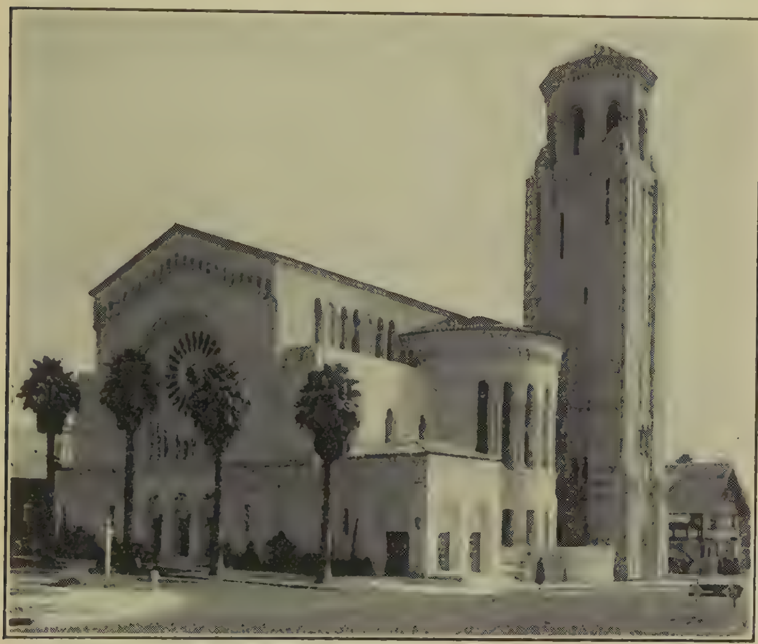

FIG. 4.—WILSHIRE BOULEVARD CHRISTIAN CHURCH, LOS ANGELES.

An example of church architecture in exposed monolithic concrete. All ornamentation, except the Rose Window, was cast as a part of the structural concrete. The surface of the entire structure was ground to a uniform finish.—Robert H. Orr, Architect.

The answer probably dates back to the early days of concrete. At the time concrete was first proposed for buildings in this country, many agencies suggested the moulding of this plastic material into stone and architectural trim. Concrete was offered as a cheap method of giving a building architectural treatment. In many cities we are still confronted with rock-faced concrete block atrocities and disintegrating examples of precast ornamentation. This early-day, over-zealous promotion has naturally made the architect skeptical and a bit careful in specifying concrete for exterior exposure. All of this promotion work was carried on before we knew very much

about the material. At that time concrete was cement, sand and stone, to which water in any convenient amount was added.

Today we can place in the hands of the architect and engineer a concrete that will fulfill his requirements as to a faithful reproduction of his

FIG. 5.—WILSHIRE BOULEVARD CHRISTIAN CHURCH, LOS ANGELES.

Entrance way at the base of campanile. All parts of the entrance detail, except the panel behind the arch were cast with the structural concrete. The wall area shown in the picture is characteristic of the finish for the entire church.—Robert H. Orr, Architect.

design and with an assurance that his design will withstand the elements at least as successfully as any other material within the economic limits of his design. In other words, the architect can accept concrete as a ma-

terial that has a quality and integrity equal to the materials he is now employing.

All of this has been brought about by intelligent and constructive research. Many laboratories have checked and rechecked the theories evolved until they agree that concrete can be made to a definite specification. Strength has been the measure of quality in the past but this feature alone does not produce a concrete suitable for architectural use. Concrete must resist the seasonal changes such as wet and dry, freezing and thawing and in some cases abrasion. Laboratory developments of the past five years make it possible to design a concrete that embodies all of these features, thus preserving the minutest detail of a design.

General Requirements.—In considering concrete for the exterior of a building there are two principal problems that must be reckoned with. First the architectural design must be one that is characteristic of the

FIG. 6.—EBELL CLUB, LOS ANGELES.

This is the Woman's Club of Los Angeles. The front elevation is characteristic of colonial design while the rear presents a Spanish patio. The entire exterior of this building is in a concrete just as it comes from the forms. All decoration is molded in the structural concrete.—Sumner Hunt and R. S. Burns, Architects.

material and second, the quality of the concrete must be such as to resist the action of the elements.

The illustrations used with this paper show conclusively that the exterior and exposed use of concrete must be a frank expression of the material. Designs to be done in concrete should not be the same as those of brick, stone or marble. Long pilasters should carry the eye from the ground to the sky line. Walls may be broken with windows, doors, arches or relief work but all should be in harmony with the general effect of the entire design. Embellishment should be so placed as to break up the large wall areas that would tend toward the unpleasant. One architect has said that, "All the lower areas of a building must have a friendly surface." In other words, these portions of the building that the public notice most should have refinement built into them. The balance of the structure may carry the rugged characteristic of the material. The entrance ways must

be drawn carefully, and the sky line may be decorated with a wealth of detail. After all, the dominant note for the concrete building should be that of a monolith that has arisen out of the ground upon which it stands. It should leave the impression that it had always stood there and like the

FIG. 7.—EBELL CLUB, LOS ANGELES.

A view of the patio. Unusually pleasing effects have been produced by careful study of form details. All molds and trim were cast in the structural concrete.—Sumner Hunt and R. S. Burns, Architects.

great Redwoods produce a picture in our mind of sturdiness sufficient to combat the elements for centuries.

The concrete for walls of this type must be of uniform composition throughout the entire area. The size and grading of the fine and coarse aggregate should be consistent throughout the job so that the concrete

will be uniform. A mixture composed of one bag of cement to six and one-half gallons of water will produce a concrete that will resist weathering and prevent the absorption of moisture into the wall. The consistency or workability of the concrete is very important. The concrete should be of such a consistency that it will go into all of the corners and angles of the forms without excessive spading, but the mix should not be so wet that

FIG. 8.—BEVERLY PROFESSIONAL BUILDING, BEVERLY HILLS, CALIF.

The design of this building is characteristic of reinforced concrete. Long, pilastered effects add height and dignity to the structure. The exterior treatment consists of a cement finish over the monolithic concrete with all spandrels cast in the structural concrete. The spandrels were stained in order to produce the contrasts.

after the concrete has been in the forms for ten or fifteen minutes water rises to the surface. Each batch of concrete should have the aggregates and water measured very carefully so that the density of every batch will be the same.

It is also important that the concrete be evenly distributed along the forms and not allowed to flow by gravity from the point of discharge to

any point in the wall. The form should be filled evenly and where possible the concrete placed in one continuous operation. Any breaks in the placing should be made on definite lines or along the bottoms of window openings.

Application to Building Types.—The construction under consideration divides itself into three separate and distinct types; (1) the multi-story building of reinforced concrete with all of the structural members includ-

FIG. 9.—PACIFIC COAST CLUB, LONG BEACH, CALIF.

An example of marked-off stucco and moulded decoration. The stone marking is made in the brown coat and each separate stone is given a color treatment in order to produce mottled effects. The crest of the club was pre-cast and set before the cement finish was applied.—Curlett and Beelman. Architects.

ing the walls of the same material placed monolithically with the columns and floor system; (2) the multi-story building with structural steel framework and reinforced-concrete enclosing walls; (3) the reinforced-concrete building six stories and less where the walls are load bearing and the spaces between the windows serving as narrow, rectangular columns. In

each of these classes the monolithic wall is the principal feature and the point of economy in the construction.

Reinforced-concrete walls have three distinct functions in buildings of this type. In the first place they replace masonry walls in ordinary con-

FIG. 10.—THE SAN FRANCISCO CHRONICLE BUILDING.

An excellent example of Gothic architecture done in monolithic concrete and molded decoration. The mullions were cast roughly in the structural concrete and run true with a cement finish.—Weeks and Day, Architects.

struction and enclose the building; second, they offer greater stability than any other form of enclosure. Such walls may be figured as taking a definite part of the normal wind stress, thus relieving the columns of part

of the stress or reducing their size. The third point, and the one that has greater consideration in this paper, is the employment of these walls as a part of the architectural scheme of decoration.

Engineers generally agree that any type of construction giving greater stability without greater cost is a desirable construction. Monolithic walls placed at the same time as the columns and floor systems give greater stability and rigidity to the building. On the Pacific coast this method does not increase the cost even in the face of ten- and twelve-dollar brick.

FIG. 11.—LAKESIDE APARTMENTS, OAKLAND, CALIF.

An example of monolithic construction with a cement finished exterior. The building is characteristic of an apartment house erected in any part of the country. It is also characteristic of designs in monolithic concrete.—Maury Dreggs, Architect.

In localities where wind storms of great severity occur or where earth shocks are an ever present menace, buildings are sometimes subjected to horizontal forces of great intensity. Such forces may be sufficiently great to severely damage or completely destroy modern skeleton frame buildings of even the sturdiest types. Walls which in themselves do not lend rigidity against such forces but which either crush or act independently of the structural elements permit differential movement between columns and beams.

Monolithic walls which are a part of the structure, however, tend to tie it together and resist horizontal stresses. Shear walls are a necessary part of any structure and since concrete walls can be provided with additional reinforcement, at slight expense, sufficient to provide full stability

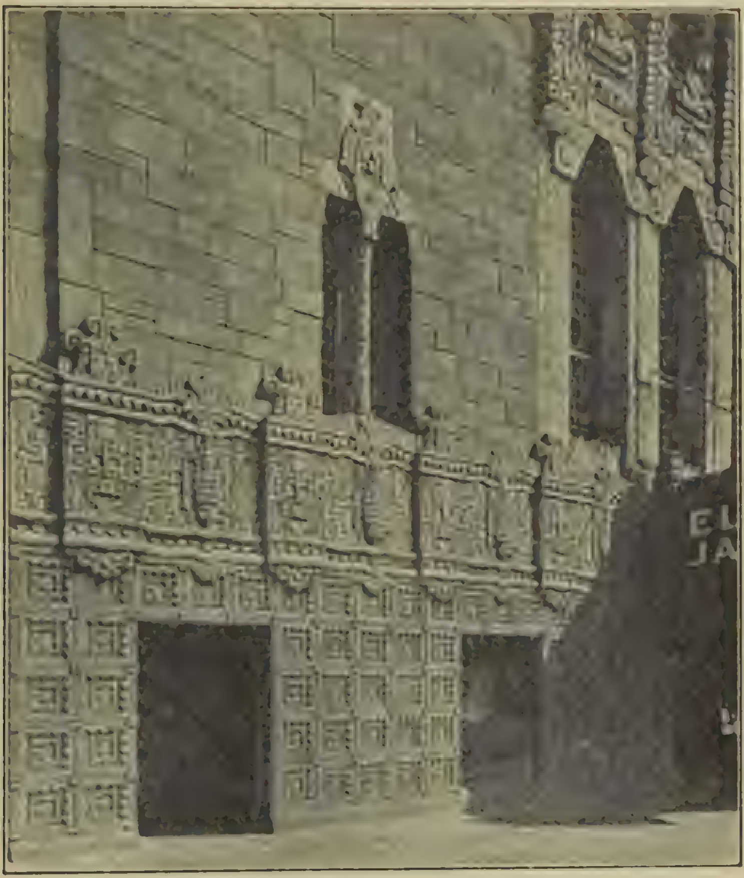

FIG. 12.—MAYHAN THEATRE, HOLLYWOOD, CALIF.

The finish of this theatre consists of molded decoration at the base and around all window openings. The ashlar work in a cement finish produces an effect similar to brown stone.

to the structure, it is only logical that the additional virtue of the monolithic wall should be put to good use in any locality where wind storms and earthquakes are problems.

Most building codes require a brick filler or panel wall to be at least 12 inches thick while the same wall in reinforced concrete need only be 8

inches. On this basis, the cost per square foot of wall area will favor concrete. Even though the concrete cost may be more it should be remembered that greater rigidity has been secured with a considerable reduction in the final load on the footings due to the fact that these filler walls also serve as a final finish of the exterior of the building. In other words, the load of a facing material need not be considered in these calculations.

A summary of interviews with some twelve or fifteen engineers, architects and contractors on the Pacific coast indicates that all favor monolithic walls over masonry construction. These men say that the cost is generally less and a greater rigidity is obtained. One contractor points out that the building goes up slower but that the total time of enclosing the structure is less.

A great many factors enter into the cost of buildings so that it is difficult to draw comparisons that are reasonably accurate, therefore the figures given below naturally involve some tolerance in either direction. It is felt, however, that the differences in cost between the two types of construction should serve as a guide. These figures have been supplied by competent contractors and represent the same class of occupancy in each case. A structural steel frame hotel with masonry walls, face brick and cut stone trim, costs 67 cents per cu. ft. A reinforced-concrete frame apartment hotel with brick panel walls and a face brick exterior cost 65 cents per cu. ft. Six apartment hotels, using a reinforced-concrete frame with monolithic panel walls and stucco exterior, vary in cost from 47 cents to 52 cents per cu. ft. Published costs for stucco treatment on monolithic walls consisting of a dash coat for bond, a brown coat for strengthening the surface and a colored finish coat varies from $1.10 to $1.50 per sq. yd. All of the above figures are based on labor costs in southern California which are considerably lower than those in the cities east of the Rocky Mountains. However, the differential between monolithic walls and masonry walls is of significance. Approximately the same relationship in costs should exist in the central part of the United States.

The practice of all western designers is to detail the construction of monolithic walls along with the columns and floors. The concrete is placed in the wall up to the soffit of the spandrel beam and allowed the customary two hours to settle. The beams, columns and floor system are then placed. This procedure makes a real monolith out of the structure so that all members, including the walls, function together.

The reinforcement for 6-in. walls usually consists of ½-in. bars on 12 in. centers, placed in the center of the wall. For an 8-in. wall two lines of ½-in. bars are placed on each face with the proper amount of fireproofing over the bars. All windows, doors or other openings must be properly protected from diagonal cracking. To eliminate these cracks two ⅝-in. bars are placed at the top and bottom of each window and over the top of each door. These bars extend into the wall at least 30 in. on either side of the opening. Further protection consists of two ⅝-in. bars placed diagonally over the corners of each opening. This practice is the rule for

the school boards in Seattle and Portland where the concrete is subjected to considerable volume change due to the great precipitation the year around; it is also followed on all work in California.

The application of reinforced-concrete walls to structural steel framework is comparatively simple. After the frame has been riveted, outside forms one story in height are set. Reinforcement is then put in place after which the inside form is set and the concrete placed through the top of the form. Such a wall prevents lateral motion under high wind and earthquake stress. These walls may serve as the final exterior treatment or be cast with grooves for veneer anchors.

The third class of structure is one in which the walls are load bearing and the spaces between the windows are narrow, rectangular columns. Such construction is particularly adaptable to schools, hospitals and apartment houses, because such walls can be constructed to a height of four to five stories without excessive thickness. There are no wall columns projecting into the rooms, thus giving the maximum usable space.

Finishing Methods.—The method of finishing the concrete walls of the buildings depends entirely upon the effect the architect desires to produce. These finishes may be divided as follows: (1) Surfaces which receive no treatment after the forms are removed; (2) surfaces similar to (1) but which have been ground down with carborundum; (3) stuccos consisting of dash coats and two and three-course work; (4) special applied finishes; and (5) veneers of brick or stone.

Surfaces that are to be left as they come from the form must be detailed carefully. The contractor must be required to use the best grade of tongue and groove lumber of 6-in. stock size. This gives an exposure to the surface of each board of approximately 5½ in. The following is an extract taken from a specification from one of the buildings used to illustrate this paper: "The forms for all exterior walls of the super structure above finish grade which are not covered by cast stone veneer shall be of ship lap lumber ⅞ in. thick. All boards shall be of the same width and laid horizontally and levelled and the joints well broken. The rough face where the surfaces are to be plastered or veneered shall be left next to the concrete and where no plaster is to be used the smooth side shall be next to the concrete."

The façade of a structure, on which no treatment other than that produced by the wooden forms, should be studied very carefully in order to determine the proper width of form boards to be used. Wide boards tend to make the structure look heavy and clumsy while narrow boards dwarf the design. The 6-in. stock board referred to above produces a desirable effect up to heights of 100 to 125 ft. The form boards should be levelled and carried entirely around the building at the proper elevation. The joints between the form boards should be so broken that they will not be conspicuous. Ends of the boards that are out of line tend to break up the architectural effect and give a displeasing surface. It is therefore essential to see that each lift of the form lumber be carried directly and

evenly around the exterior surface and well nailed. As soon as the forms are removed the walls should be brushed down with a wire brush, thus removing all fins and construction dirt, giving the surface a clean appearance. The forms must not be painted with oil or other substances as this treatment will cause discoloration and prevent the bond of plasters or stucco.

It has been found by practice that it is economical to grind this surface so as to remove all projections and give the structure a smooth appearance. This grinding is done with a portable electric motor and carborundum wheel. The grinding does not remove all of the form marks but merely evens the surface giving it a smoother appearance. The forms in this case should be designed for definite heights and filled with concrete to definite levels so that when they are stripped the effect will be uniform throughout the structure.

The application of stuccos to monolithic walls makes it possible to develop a great variety of architectural effects. There has been some hesitancy on the part of architects to specify the application of stuccos or plaster on monolithic concrete walls. This method of treatment is followed extensively on the Pacific coast and has proved entirely satisfactory. It is recommended that one of the two procedures given below, be followed carefully for this same class of construction in the central and eastern part of the United States.

A special material is now on the market, that may be applied to the exterior face of the form, that delays the hardening of the concrete against the form. After the forms are removed the surface of the concrete can be brushed to a depth of 3/16 in., thus exposing the aggregate and providing an excellent mechanical bond for the plaster. The wall is then scratched, browned and finished in the procedure recommended for such practice. The other method and the one that has proven successful on the west coast is to dash the surface after the concrete has hardened and dried out. The dashing covers approximately 60 per cent of the surface and provides an even suction throughout the face of the wall for the brown coat. This dash consists of a mortar mixed to a creamy consistency in the proportions of one part of cement to two parts of sand. It is thrown forcibly on the wall with a dash brush. Quite often this dashing is done in color and allowed to stand as the final finish for the structure. In other cases it serves as a bond coat upon which is applied either a two- or three-coat stucco. In three-coat stucco, broken ashlar or marked off stone effects can be produced that are very acceptable as finishes. The finish of the separate stones can be varied in color-tone so as to produce mottled effects. Several illustrations are shown of this completed work.

Another method of applying finishes to monolithic walls is by using the cement gun. Scratch and brown coats can be built up and a finish coat in color applied by hand. Where mottled effects are desired, a brown coat in color may be shot on with the gun. Past experience in this method of application indicates that it is impossible to get an even color over a

structure where the finish coat is applied with the cement gun, due no doubt in a large measure to the variation in suction in the scratch coat under this method of application. The suction or amount of water absorbed from a colored cement stucco mix controls the final color effects.

A fourth method of finish consists in the application of specially prepared plaster or precast ornamentation. I have particular reference to the type of finish used on the Parthenon at Nashville, Tennessee. This finish is applied either under the same methods as plastering or by casting and applying to the surface of the wall, a facing composed of special colored aggregates. After placing this material the aggregates are exposed by a dilute muriatic acid wash. This method produces concrete that is very hard and durable and permits the carrying of the most minute details of moulds, columns, flutes and bas relief.

The greatest advantage to be gained by the use of monolithic walls is the possibility of making the architectural decoration an integral part of the wall. By this method the contractor builds reverse plaster moulds into his wood forms at the proper place. These moulds are filled at the same time and with the same concrete as used in the balance of the structure. As a result when the walls are complete all of the architectural decoration is in place, thus saving the time of scaffling and the item of extra labor for setting stone. Do not confuse this method with precast trim. Perhaps the method should be called moulded decoration. After the forms have been removed the plaster moulds are left in place until the building is practically completed. This time interval permits the concrete to harden slowly, thus gaining its maximum strength. The moulds are then removed by breaking up or prying off, which operation completely destroys the plaster of paris in all undercuts. The time interval above referred to also permits the concrete to gain sufficient strength so that the plaster moulds may be broken off without damaging the surface of the concrete. After the moulds are removed, the decoration is cleaned with wire brushes and colored if desired with thin cement dashes, oxides of iron or chemical stains.

An interesting method of filling the forms in and about plaster moulds has been developed. Before the concreting operation starts, these moulds are covered with canvas to protect them from splash. After the concrete reaches a point that is well above the center of the mould, the canvas is withdrawn which permits the concrete to flow back against the mould at one time. This procedure gives the best results and prevents scaling or pock-marked surfaces. It is highly recommended and one that should be used whenever this method of decoration is employed.

Concrete has another application in the treatment of buildings. In the past it has been the practice to plaster and decorate the lobbies, lounging rooms of hotels and clubs, interiors of theatres and elevator lobbies of large buildings. Many architects have realized the possibility of including the structural members of a building into the architectural or decorative scheme. In other words concrete beams and girders may be so designed

and built as to become an integral part of the decoration. In designing the ceiling of a lobby or lounging room in which the structural members are to be stained, it should be realized before the concrete is placed that the final effect will depend largely upon the quality of the surface on which the decoration is to be applied. These concrete surfaces should be free from blemishes and the forms should be erected rigid enough to support the loads imposed without swelling or buckling. Textures can be produced in the surfaces by the use of undressed lumber for the forms or by means of specially prepared plaster moulds.

Before applying stains and paints to the concrete, the surface must be neutralized to prevent saponification. After the concrete has dried for a period of six to eight weeks the surface should be treated with an application of zinc sulphate. The next step is the application of a priming coat which practically fills the pores of the concrete and prevents the stain or oil of the paint, from penetrating too deeply. The priming coat consists of an application of a boiled linseed oil. If the concrete is very dense it may be necessary to add a small percentage of turpentine to the linseed oil in order to secure the proper amount of penetration.

It is essential that the identity of the concrete be not lost by means of a paint film. In other words the texture or characteristic of concrete must predominate after the decoration has been completed. It is therefore suggested that the surface of the concrete be stained rather than painted. Stains consisting of a mixture of boiled linseed oil and chinawood oil thinned with turpentine or naphtha and colored with pigments will give a satisfactory surface and permanent stain effect.

After the concrete has been primed and stained, it can be painted with any good oil paint used for stenciling decoration. By following these instructions the most delicate colors may be used and not affected by the concrete. A further protection to the decoration is a starch surface which will protect and preserve the stenciled decoration.

Stenciled designs on ancient walls and ceilings, sacred to the architect and artist, can be revived and placed in our modern building with an assurance that they will be economical and as permanent as the ancient examples from which they were drawn. Concrete has opened a new field of building decorations both interior and exterior. Modern processes of making concrete exercise no restraint upon the architect, but on the contrary this material affords a greater freedom in design than any other material.

A late issue of one of the leading architectural magazines carries an editorial on a new architecture that might well be quoted here.

> "America is standing on the threshold of a doorway opening to a new architecture. No moment in all the past has been so bursting with promise as the present. Our architects have had more money to spend in new construction than the architects of any time or any land. Unlike any previous epoch, this money has been available not for the mere embellishment of buildings, not for the glorification of some wealthy patron of the arts, as in the high tide of the Renaissance in Italy, but rather than through efficient plan and construction the money spent might earn more money.

* * * * * * *

"It is surprising that under these new and ever-changing demands the old formulas are being found inadequate? Twenty-five years ago we were asking, somewhat hopelessly: Are we ever going to develop an architectural style in America that will belong to it as the Gothic cathedral belonged to France of the Middle Ages, or as the Parthenon belonged to the Greece of the Golden Age?

"We have but to look about us to see what the architectural style which we were unable consciously to create is here with us today. It is still a bit wobbly on its legs, not sure of itself, just entering into individual life, not fully satisfying us, but it is going forward."

Concrete might well serve as the basic material from which to build an American Architecture. It is a building material equal to any other building material known and far superior to most building materials now used. It has the enduring characteristics of stone, it can be moulded and fashioned, and late developments convince us that it can be finished with any color or texture desired. It can be carved, stained or painted as its use in the building may require. In other words, it is a universal building material because it possesses the three properties required of a building material, i. e., structural strength, fireproof and weather resistant properties and decorative possibilities.

DISCUSSION.—REINFORCED-CONCRETE WALLS FOR BUILDINGS.

Mr. Bright. JOHN IRWIN BRIGHT.—In commenting on Mr. Hart's paper I will do so from the viewpoint of the architect, the one whose duty it is to select the best building materials under the varying circumstances of cost, strength and appearance. In these matters his profession demands that his interest be general rather than special, while with equal propriety Mr. Hart's chief concern is the economic and technical advancement of the cement industry.

It is evident that our approaches to the problem of the advantageous use of cement, while equally correct, are, nevertheless, different and hence the conclusions reached by proponents of each method will necessarily be dissimilar.

The behaviorism of the architect can best be understood by a definition of the word. At its very root lie the inseparable ideas of artist, of chief artificer, master builder and workman. To men of this stamp we owe the Acropolis of Athens. Webster says that an architect is not only a designer but is also one skilled in the art of building. He must be equally interested in structure and asthetics. But throughout Mr. Hart's paper the architect is described as one who applies ornament to a surface fashioned by others, and whose dearest wish is for a medium, which by reason of its inherent properties and its low cost will permit the use of decoration in hitherto undreamed quantities.

It is quite true that the architect is often held to a budget too small to permit the ideal development of the scheme, in which case many things suffer, including the element of decoration. But the well-trained architect is not unduly worried about embellishment. He regards it as just one of those things which perforce must take its proper and unobtrusive rank in the welter of financial, structural and artistic claims.

The architect, therefore, must be scientifically minded, attentive to the merits of every material, making his choice with a sole view to appropriateness in the structural, artistic and eonomic pattern. In other words, the selection between two or more building mediums depends upon their relative values always considered in the light of a specific problem. And the value of any material is always the balance between its virtues and defects. When concrete is adopted under these safeguards one may have a reasonable expectation of good results.

Our technical knowledge of concrete is by no means final. There is still some uncertainty as to how it will react under different conditions of handling, mix and temperature. True it can be placed by unskilled labor and yet the greater the economic advantage from this source the more the risk in the technique of fabrication. We are in almost complete control over the physical influences surrounding the manufacture of steel and brick, and as an additional surety it is possible to scrutinize and test every last piece of metal and burnt clay before its incorporation in the structure. But with concrete it is different. During the brief hour of its forma-

tion almost any accident may befall it. It can be mechanically tested only as a constituent part of a completed edifice for the very good reason that prior to its integration in the building it did not exist. A few days after a soggy slush has been dumped in the mould it emerges hard, naked and unashamed. The die has been cast and it is then a matter of accepting it for better or for worse. All these are handicaps, no doubt, comparable to those which pursue us in our human relationships, but in my opinion they are not formidable enough to cause its rejection by a designer with a taste for adventure in his blood. However, there is little need for alarm, for if ordinary precautions are taken the risk of complete failure is almost non-existent and even the minor imperfections tend to become of less and less importance as our working experience broadens. The engineer is taking all the fun out of it.

On the other hand, concrete has many unique merits in the fields of structure and appearance. If an advantageous and characteristic result can be obtained from its use then it is quite as immoral to substitute for it any other material as it is to subject fine, honest concrete to a debasing manipulation. Any attempt to use it imitatively is opening the door to artistic bankruptcy. Grinding, hammering or brushing the surface may be advisable under certain circumstances, but it must be remembered that arrises are easily damaged and that firmness of contour once lost is difficult of recapture. The natural color of concrete cast in wood forms is extremely beautiful. Its very uneveness of tone and texture lends it charm, and any surface treatment destroys these tones and substitutes others with sometimes a loss of artistic effect. These few suggestions are not intended as a guide to practice. They are only a plea for simplicity and directness of thought.

Mr. Hart is an enthusiastic advocate for concrete. While frankness compels me to state that I feel that some of his claims are lacking in, let us say, a factor of safety, I can at the same time deplore the hesitancy of architects to experiment with this alluring material and thus circumscribe their artistic development and their usefulness to their clients. It has its limitations. Of this the Romans were well aware. Much of their concrete exists to this day, but it is generally of mass construction where stone masonry would have been uneconomic. In spite of the fact that we moderns have learned how to increase its tensile strength by the incorporation of steel bars in its mass, its application is still restricted, and the denial of this fact serves no useful purpose. Such excess of zeal does positive harm and retards the cause which it is intended to serve.

Personally I love concrete, although it has not always been faithful and true to me. But its seeming treachery may have been a just retribution for my lack of skill and understanding rather than because of any inherent vice. My slight experience has convinced me beyond question of its great economic and artistic possibilities and that it is both desirable and proper that architects should be urged to seriously examine its potential value as a building material.

Mr. Johnson. VIRGIL L. JOHNSON.—Mr. Hart has given us a very admirable paper. Many interesting arguments have been brought forth showing why concrete should be used throughout the entire structure. In fact, so numerous are the suggestions and arguments that one may be at a loss to know what system should be followed. Shall we have the surface exposed or shall we cover it up with stucco, plaster or paint? If exposed, shall we grind off the edges or leave it in the rough? Or shall we follow the more costly method and introduce colored aggregates, glass and precious stones?

In place of trying to answer these questions, suppose we direct our attention for the moment to another phase of the subject, our main street which to a Philadelphian is none other than Chestnut St. This famous old street, extending from the Delaware River for a distance of approximately ten miles, is lined on both sides with block after block of buildings. Today, as of yore, the most beautiful and the most architectural of all is the group located at Sixth and Chestnut St., in the center of which stands Independence Hall, the noblest and the best example of American Colonial architecture. As we linger to admire and to analyze this most wonderful group of buildings we are confronted with this pertinent question. After all, what is architecture? Is it something tangible to be obtained by a formula or is it acquired through a supernatural process of thought or reasoning?

Mr. Hart has very wisely suggested that architecture is one of the principal problems to be considered in the use of concrete for the exterior of a building, for he has frankly stated that "architectural design must be one that is characteristic of the material."

Now let us approach the subject of architectural design through the following method of reasoning. Some one has defined art as a beautiful thought, made visible to us through architecture, painting, sculpture, etc., or to make it shorter, it is the *visible expression of a beautiful thought.* This being so, then the idea or thought when expressed in the form of a building becomes architecture.

In the old State House, the designer, Andrew Hamilton, has expressed the thought of a seat of government of a prosperous British colony, and later it becomes the birthplace of a democracy and a new nation. Independence Hall has been brought to your attention as an outstanding example of a beautiful thought made visible to us through architecture. By its location and historical associations it has so impressed itself upon the American people that its expression is almost audible.

In the design of a building the architect must consider the site or location, its size, mass, proportion, heights, widths, sizes of openings and the relation of the openings to wall surface. Above everything else the building must express its purpose. The wall surfaces are properly considered, but they too must necessarily assist in the complete expression of the designer's thought.

This visible expression by means of any structure other than a building is plausible and in like manner becomes art either good or bad, depend-

ing completely on the designer's ability. Thus the engineer has the same opportunity as the architect in making his structure, express a thought; he can also make it beautiful and attractive providing he can properly express the idea. And, as a splendid example of a well-designed engineering structure, the Walnut Lane Bridge, over the Wissahickon Creek, is unsurpassed. For many years as you know it was the longest single span concrete arch in the world. The grandeur and the impression which this masterpiece of engineering creates in the mind of the poet has best been expressed by the words of Christopher Morley which deserves to be quoted: "Leaping high in the air from the very domes of the trees, curving in a sheer smooth superb span that catches the last western light in its concrete flanks, it flashes across the darkened valley as nobly as an old Roman viaduct of southern France." This bridge may truly be called a work of art because it fully expresses a thought and its purpose, that of a great viaduct.

If this be a digression, it may serve to show what an architect or an engineer can do to express a thought in the form of a structure. In doing this, we hope to answer Mr. Hart's question, "Why do not architects use more concrete on the outside of their buildings?" We believe that architects in general will employ a concrete surface when and where they find that they can use it as a means of expressing the purpose of the building and the thought of the architect. Furthermore, they will probably overlook the cracks, crazing and other characteristics of concrete providing it is in keeping with the general design of the building.

When the concrete foreman, the engineer and the contractor co-operate and produce a poured-in-place concrete with a dependable surface which can withstand the elements of the North Atlantic coast, not one year, but for five or more years and still be attractive, then and not only then will the architects feel free to adopt its use. The architect in seeking material for his building likes to use that which if condemned, may be taken out and rebuilt to satisfaction. It is next to impossible to make a contractor remove a portion of a concrete wall which shows the ordinary defects and characteristics of a poured-in-place concrete such as air pockets, crazing, exposed reinforcing rods and honeycombing. There is no use deceiving yourself into thinking that you are going to get a 100 per cent perfect surface when you are in such a position that you are expected to accept the lowest bidder.

On the average, it takes one to two years to educate some contractors to get 2000-lb. concrete and we feel sure that it would take five years to produce the kind of surface that an architect would tolerate.

On the other hand, if the architect had the privilege of choosing his contractor he might get the desired surface every time. One of the great drawbacks to the architect using concrete is the necessity of constant supervision which must be given throughout its placing and curing. Because of this constant supervision the architect becomes a craftsman or a manufacturer as well as the architect. This argument proves conclusively that

in order to carry out the wishes of the architect, there must be more and more trained concrete foremen. Foremen that will give ideal concrete rather than those who are trying to save money for the boss.

Mr. Hart has very wisely stated that "Designs to be done in concrete should not be the same as those of brick, stone or marble." It is quite impossible to think of a concrete surface embellished with the delicate carving of the Italian, or French Renaissance or even the Gothic style. You will notice that the architect is obliged not only to satisfy his client but he must meet the criticism of the public; thus he selects his material with caution.

Mr. Hart goes into the details of the monolithic walls even to the reinforcing but does not state how to eliminate the condensation on the inner surface of the wall. His paper has pointed out the splendid advantages of rigidity that the concrete exterior wall gives to a building. This is a quality that should be emphasized, for a building with such walls presents an ideal protection not only against fire, but against the elements such as windstorms, tornadoes and earthquakes.

It is possible in the future as a new style of architecture is developed that concrete will be extremely useful to the architect in the expression of his art.

Mr. Powers.

E. S. POWERS.—The ordinary method of placing ties that will show rust spots on the surface, I think will be objectionable. We have used spools, but that is so expensive in most of the work that I was wondering if there was any other method being developed? We usually allow 2 in., but it has been found insufficient.

Mr. Hart.

W. E. HART.—Wherever it is possible to brace on the outside, that is done. Otherwise they may use the wires through the forms or in some cases a form spacer. The latter is a piece of pipe on which the outside lags unscrew so that it may be filled with the mortar from a regular batch, giving practically the same color and texture.

Mr. Perrot.

E. G. PERROT.—I was very much interested in the question that was raised about the difficulty of keeping the iron reinforcement from getting too close to the surface and thereby causing rust. I would like to know just what they are doing in the West to obviate this particular difficulty. We do not seem to have such artisans here in the East as they have in the West to make satisfactory forms. In fact, we have abandoned building concrete walls because we can build the veneer of brick or stone much more satisfactorily and eliminate the danger of having the concrete disintegrate due to the rusting of the reinforcement.

Mr. Powers.

E. S. POWERS.—In erecting a factory building, the scarcity of building material caused us to do just what has been recommended here—make a building of exposed concrete. One of the problems involved was the support of about four floors on a 63-ft. girder. We could not get any steel nor could we get reinforcing bars that were as long as the girders. We decided to take a chance and build the girder as a rectangular beam, 12 ft. deep and 4 ft. wide. I have observed in the last few years the appearance of

rust on the face of that important girder. We used all the skill we had at our disposal at that time in placing both the concrete and the reinforcement, but it is now being revealed that there is trouble brewing. I am in sympathy with the movement to use concrete in walls, but such examples as this are deterring myself and a good many others from recommending it to our clients.

NATHAN C. JOHNSON.—There is a great deal of surface rusting of reinforcement visible. I attribute this largely to two major causes. The first one is the wrong conception of concrete as needing such an awful lot of steel for alleged temperature or shrinkage or something else, so that in a narrow wall, say 12 in. thick, 2 rows of steel rather close to the surface are quite commonly required, whereas one row at the center would be entirely sufficient except for bending. In fact, the less steel you have, within a certain range, the better off you are. **Mr. Johnson.**

The second cause lies in the so-called "practical" procedures wherein and whereby men do not brace their steel, but lay it up against the forms, and then as the concrete rises, take their shovels and pry it back, thus permitting a little "gravey" to run around on the outside. Then, when the forms are stripped, of course the steel is hidden, but as I will endeavor to show you when I speak a day or so hence, the concrete is so permeable, even the best of it, that it is very responsive to atmospheric tides. By that I mean that in a moist day, there is a very pronounced taking up of water, and in a dry day there is a giving off of water, and of course, while that water is resident in there, it can do its damaging work on the steel.

Another thing that causes trouble is that it is quite customary and "practical," to use that much abused word, to put in steel that is badly rusted. If you should get that rust off, you would find that it was very deep. Rust is a kind of cancerous affair andwill go deeper, once it has an origin. It is very interesting to see it. Some day I hope I may show you some of the motion pictures taken of steel rusting; it is very much like the blooming of a flower.

Those three reasons account for a great deal of the surface disfiguration referred to in this paper.

W. A. SLATER.—I certainly believe that we do not want to use more steel in a concrete beam than necessary, but some large beams demand a great deal. There are some instances that have shown that steel can be placed in concrete and be prevented from severe corrosion, even under severe conditions. I had the privilege of looking at one of the concrete ships built 7 or 8 years ago. The shells of the concrete ships were 4 to 5 in. thick and there was a great deal of reinforcement in them, never more than half an inch from the surface. As I examined the concrete ship just above the water level, there was very little evidence that there had been corrosion of any importance. Of course a rich concrete had been used, and this may be one of the necessities to combat corrosion in places where it is very important that it should be prevented. **Prof. Slater.**

Mr. Hart. W. E. HART.—The important thing I tried to point out in this paper is that, to begin with, it is essential that you have a concrete that will resist the weather. I would say that at least 90 per cent of the difficulty with rusting of bars is due to our old time mixes. If you would analyze those mixes on the basis of dry materials, you will find that instead of having a 1: 2: 4, you have a 1: 1½: 4. In other words, you have a harsh mix that will hardly resist the weather. My recommendation is 6 to 6½ gal. of water where the concrete is exposed to weathering conditions. By this means it is entirely feasible and possible to prevent rusting in bars 1 in. under the surface. The practice on the West Coast on 6-in. filler walls is to use ½-in. bars on 12-in. centers in the center of the wall. As a general rule the reinforcement is put in the center of the wall.

Mr. Price. P. W. PRICE.—For the surfaces that are left just as they come from the form, are any precautions taken in regard to horizontal contraction joints?

Mr. Hart. W. E. HART.—The practice generally is to use 2 x 4 in. studs varying in length from 14 to 20 ft., erected in a staggered relationship. Form boards are then layed on the inside of the studs, thus making the forms continuous. This method eliminates longitudinal joints in the forms when necessary to stop placing concrete. It is either at the floor level or under the window-sill line.

Mr. Gresecke. F. E. GRESECKE.—I think the concrete has very much to do with the rusting and non-rusting of steel. We built a tank with walls 4 in. thick, and it had to be broken up. In breaking it up we exposed the steel and it was perfectly bright after several years' use.

A Method for Predicting Concrete Strengths With Increased Precision.

By Herbert J. Gilkey.*

INTRODUCTION.

While great progress has been made in the pre-determination of concrete strengths, even the most optimistic friend of concrete must concede, in his sober moments, that our very carefully aimed shots are still prone to fall rather wide of the mark. In the majority of cases, the great achievement thus far has been the fairly certain setting of a minimum strength for the concrete to be used in our structures. We have felt satisfied, or even pleased, if the job samples proved to be 20, 50, or even 100 per cent stronger than the pre-selected strength for which the mixture was designed. This is a very natural feeling but it is more that of the layman than of the engineer, whose function it is to build safely upon the one hand and economically upon the other.

Viewed dispassionately, if 2,000 lb. concrete has been selected as the needed mixture, 1,000 lb. concrete is a hazard and 3,000 lb. concrete an extravagance.

Without regard to whether the basic law, by which the strength of concrete mixtures can be predicted, has been found, or whether or not any such law will ever be found, the present paper purposes to show how essentially the same result can be obtained by using more efficiently the facts which are already at our disposal. No existing theory of proportioning concrete is to be attacked and no new theory is to be proposed. Nevertheless a slight change in the technique of application, of whatever theory we have been accustomed to use, will serve in most cases to lower the spread between the expected strength and that obtained from disconcerting values often exceeding 100 per cent, under even the most favorable conditions, to a nominal figure comparable with the most uniform of structural materials.

Moreover there has been a growing feeling, on the part of the practical man, that the language of concrete quality and strengths has little of meaning for him.[1 2] What does 2,000 lb. concrete moist cured for 28 days at 70 deg. F. mean in terms of the pavement or the building, subject to any temperature (above freezing, let us hope) and which may be artificially moistened for from zero to fourteen days? Moreover in service it may be in almost any state of semi-dryness.

*Associate Professor of Civil Engineering, University of Colorado.
[1] A. R. Lord, *Proceedings*, A. C. I., 1927, pp. 487-8.
[2] P. H. Bates, *Proceedings*, A. C. I., 1927, pp. 488.

It is easy to understand the lack of respect, sometimes bordering upon scorn, which the practical man has for much that emanates from the laboratory. One can appreciate his occasional insistence upon "job control"[3] rather than "quality control"[3] specimens. The quality control specimen is taken from the job concrete but is cured and tested in a laboratory under controlled standard conditions. It is intended to be a check upon the quality of the concrete as it left the mixer but reflects nothing of the subsequent history. The job control is presumably stored and cured with the structure and represents the actual strength of concrete in the structure at the time of test. Both sets of information are very desirable but the job control is particularly appealing. The drawback to the job control specimen lies in the fact that it rarely even approximates the concrete of the job. Adequate job control is usually very difficult to attain and test results purporting to be indications of actual strength or quality are likely to be very misleading. Due to differences in size, surface-volume relation, exposure to drafts, curing moisture, differences in retention of the chemical heat of setting, form protection, etc., the moisture and temperature status of the job control specimen is likely to differ greatly from the nearby concrete that it supposedly duplicates. Thus when unusual facilities are not available for attaining good job control, it is probably the part of wisdom to concentrate any extra effort on better quality control specimens. Quality control should always be the major type of specimen. If adequate facilities are available, auxiliary job control specimens are much to be desired. At present it is only the unusual project that can hope to have a good and adequate job control record. It is therefore highly desirable that there be some means of making the quality control specimens tell something about the probable job strengths. To point out a rational approach to this problem is one of the purposes of this paper.

There is much that we don't know about concrete, and much more that we know but imperfectly or in part. In like manner much that we think we know is doubtless misinformation that soon we shall be endeavoring to unlearn (which is always much harder than learning).

In attempting to put in shape for current use some of the imperfect information that has thus far been left more or less upon a high shelf, the writer is fully aware that he is offering material much of which is half-baked and some of which is probably fallacious. The writer has no illusions upon this point. He does feel that these incomplete tid-bits of knowledge should be placed in service, even though there must of necessity be subsequent retractions and modifications.

So far only occasionally has it been attempted to cantilever from the known to the unknown in concrete strength prediction. We have, for the most part thought in terms of one variable and almost never has any analysis been attempted simultaneously for more than two. Many will feel, and rightly so, that we have no adequate experimental background for

[3] *Proceedings*, A. C. I., 1927, pp. 55 and 70.

attempting it now. Nevertheless, the directing of thought and development of a method is not premature, and imperfect constants, or too sweeping generalizations, will give more precision than is now obtained by ignoring all factors that we do not feel to be entirely or satisfyingly mastered. It is the purpose to try to bridge the gap from the known strength of one mixture (from given cement and aggregate) for a given curing temperature, water-cement ratio, period of moist curing and time of mixing, to a reasonable approximation of the strength of a similar mixture for different water-cement ratio, curing temperature, curing age, and time of mixing.

The treatment will consist of:

1. *Presentation of the Problem* including a partial analysis of certain of the leading factors bearing upon the strength and quality of concrete.

2. *The Explanation of the Method.*

3. *Tables of Factors and Their Use.*

4. *Explanation of the Figures*, followed by a *Summary* and *Acknowledgments.*

1. PRESENTATION OF THE PROBLEM.

The quality of concrete, regardless of what particular test may be used as its measure, is influenced by many factors that may be more or less arbitrarily classified as major and minor.

Major factors are:

(a) The cement

(b) Ratio of mixing water to the cement.

(c) Ratio of voids to the cement.

(d) Effective age, i. e., period of moist curing.

(e) Curing temperature.

Minor factors:

(f) Kind, quantity and grading of aggregates.

(g) Time of mixing.

(h) Incidental factors and personal equation.

Factors (f), (g) and (h) can only be considered minor within the range of usual accepted practice. Extreme deviation from usual materials, grading, mixing or procedure would elevate any one of them to major importance.

(a) At present the cement is probably the most important of all the usual factors. The products of different mills vary greatly even though all pass the standard tests for portland cement. The report of Committee C-1 (Cement) of the American Society for Testing Materials on the co-operative tests on 32 different cements, conducted by 52 laboratories will doubtless show that the cement itself may double or halve the strength of mortars or concretes in many instances. This is but the writer's prediction based upon observations in his own laboratory. Not only do different standard cements vary greatly, but the product of the same mill may

fluctuate widely at times.[1] The same cement varies greatly with age and the extent and rate of variation is much influenced by the kind of storage.[2]

(b) The significance of the water-cement ratio needs no comment. (See Tables 2 and 3.) Its importance cannot be overestimated but the prominence given to it during the past ten years has had a tendency to make us overlook or underrate other important factors.

(c) The voids-cement ratio[3] is closely allied to water-cement ratio due to the important part that water plays in influencing the voids of the mortar or the concrete. The details or relative merits of the two ratios need not be here considered. For the present purposes they may be considered as alternate methods for designing concrete mixtures of pre-determined strength.

(d) Effective age or period of moist curing[4] bears a most important relationship to the quality of the concrete. (See Table 4.) In the dry state concrete is practically dormant so far as increase in strength is concerned. Apparent contradiction of this [5] is often observed in concrete that gradually increases in strength when exposed to the air for long periods. This may be explained on the basis of a slowly continuing hydration by the extraction of moisture from the air. It is probable that the phenomenon would not appear in very dry air.

It has also been observed by some[6] that moist cured concrete seems to retrogress in strength after long periods of exposure to relatively dry air.

This may possibly be explained on the basis of a slow loss of water of crystallization under long exposure. This belief would seem to be supported by the fact that even relatively low heating temperatures cause falling off of strength.[7] Concrete, however cured, is stronger in the air-dry

[1] Concrete (Cement Mill Ed.) Aug., 1927, p. 36 (R. T. Giles); Concrete (Cement Mill Ed.) July, 1927, p. 42 (Bureau of Standards).

[2] Portland Cement Association, Lewis Institute, Bulletin No. 6.

[3] See Bulletin 137, University of Illinois Engineering Experiment Station (Talbot and Richart), for exposition of Voids-Cement Ratio.

[4] *Proceedings*, A. C. I., 1926, pp. 395-436; *Proceedings*, A. S. C. E., Jan., 1927, pp. 79-93; *Transactions*, A. S. C. E., Vol. 91 (1927), pp. 153-167; Bulletin No. 2, Portland Cement Association, Lewis Institute (1919).

[5] P. H. Bates, "Longtime Tests of High Magnesia Portland Cements," *Proceedings*, A. S. T. M., 1927.

[6] P. J. Freeman, *Engineering News-Record*, Dec. 1, 1927, pp. 879-880. Freeman's data seem to indicate some factor beyond extraction of moisture from the air and mere drying since specimens removed from moist storage at 35 days made very substantial gains in strength up to the age of one year. It seems probable that the 60-day tests would have included all the drying increment of strength and that there would be no increase thereafter unless the air were very moist. There was, however, decided increase, more than moisture extraction from the air would seem adequately to explain. The very slow retrogression after one year agrees with Bates' experience and can be explained, perhaps, on the basis of partial dehydration, as mentioned above. The gain in strength upon placing out-of-doors is easily explainable on the basis of resumed curing. Rain, snow, fog, dew and any other form of moisture or high humidity would accomplish much added curing over a period of years. At such a location as Pittsburgh warmth and humidity might even explain the indoor strength gain up to one year but there is no apparent reason for this rather heavy gain, and succeeding smaller loss, without any apparent change in curing environment. This is a point that will bear investigation and that may necessitate some modification of statements made in this paper and elsewhere regarding certain phases of the curing process. So far as essential accuracy, for the purpose in hand, is concerned, the statements made may be accepted as correct.

[7] Bulletin No. 43, University of Washington Engineering Experiment Station (A. L. Miller and H. F. Faulkner).

state than in the wet state. By air-dry state is meant air dried to approximately constant weight which condition can be attained at most places in from one to three weeks exposure to the air, dependent upon the volume-surface relation of the concrete mass, dryness and temperature of the air in question, and whether it be still air or in circulation. Forced, or oven, drying at even low temperatures weakens the concrete as much or more than the mechanical hardening due to drying strengthens it.[7] The extra strength resulting from air drying can be superimposed upon that due to hydration, or curing, but is independent of the quality of the concrete and should not be confused with it. Subsequent soaking for a few hours or a day causes any given concrete to revert to the strength that it had in the wet condition. Thus strength gained from hydration is a proper criterion of quality but that from drying is but a temporary acquisition to be lost in case the concrete again becomes wet. It is independent of concrete quality. The point to be noted in the present connection is that the age of the concrete alone is not allied to quality (since quality gain is at or near a standstill in dry concrete) but that the age over which the concrete has had moisture available for hydration bears an important relationship to quality and most test results show that concrete will gain strength (and quality) at a diminishing rate for an indefinite period if moisture for continued hydration be available.

(e) The curing temperature is in importance on a par with moisture for hydration. In the vicinity of freezing, strength gain is slow (see Table 5), while the rate of quality gain with age increases steadily with curing temperature within any natural range. It must be remembered, of course, that no amount of favorable curing temperature will aid hydration in the absence of moisture. Both must be simultaneously present.

(f) Kind, quantity and grading of aggregates, is (within the range of usual aggregates and workable mixtures), in debatable territory as regards effect upon strength or quality of the concrete.[8] All will concede its importance as regards economy. The writer is inclined to classify (f) as one of the most important of the so-called minor factors. The correct status is not vital to the present treatment and (f) may be included or excluded as desired.

(g) Time of mixing is more important in its relation to early strength or wear, homogeneity, watertightness and workability than it is to the strength at later ages.[9] Table 1 gives an indication of the range to

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[8] *Proceedings*, A. C. I., 1927, pp. 363-414.

[9] The apparent fact that extra mixing affects early strengths more than later ones may be explained, perhaps, by some such reasoning as follows: The continued agitation in the presence of moisture may increase the rate of hydration by facilitating the penetration of the water into the interior of the cement particles. This could apply especially to the larger particles for which penetration would normally be very low. In case there is a softening of the surface, as hydration proceeds inward, as some claim, the softened exterior might be abraided in the mixing and fresh cement substance exposed to contact with the moisture. This might be either the main cause or a contributing one, with others. In any event it is purely speculative and not to be rated as more than an unchecked idea, the expression of which is justified only because it might be suggestive to some qualified investigator.

be expected. The tests from which these data are derived were performed in 1918 or earlier and there has, of course, been considerable development in mixing equipment and methods. Since time, and not speed of mixing, seemed to be the important factor it is not likely that the status has been greatly influenced by the evolution of mixing machinery. From the data available the following summary should define sound conservative practice with due regard to the element of economy:

(1) Never mix less than one full minute after all ingredients are in the mixer.

(2) When extra quality concrete is desired but early strengths (7 days or under) are not of prime importance a two-minute mix is justified.

(3) If high early strength is important, mix for five minutes. This may increase the early strengths 50 per cent or more and will slightly improve the quality of the concrete at ages of 28 days and greater.

(4) Mixing beyond five minutes is justified only when it is desired to gain the last possible increment of early strength. Probably measurable gain would rarely if ever result from mixing for more than ten minutes.

The above summary and the data of Table 1 are based upon the Portland Cement Association published data on the subject.[10] Recently the Bureau of Public Roads has been collecting evidence upon the subject in connection with extensive field tests under the dirction of Mr. J. L. Harrison.

(h) Incidental factors and personal equation. Under this heading come such items as thoroughness of placing, segregation, water gain,[11] etc.

The foregoing eight factors (a-h incl.) may be divided into two classifications that might be termed: (1) *Systematic factors*, or those that bear a more or less well-defined or orderly relationship to strength or quality of the concrete; (2) *chance factors*, or ones which at present, at least, are haphazard or happen-so as regards effect upon concrete quality. They may be important but nature and extent of effects are difficult to anticipate.

(1) *Systematic Factors*

- (b) Ratio of mixing water to cement.
- (c) Ratio of voids to cement.
- (d) Effective age or period of moist curing.
- (e) Curing temperature.
- (g) Time of mixing.

(2) *Chance Factors*

- (a) The cement.
- (f) Kind, quantity and grading of aggregates.
- (h) Incidental factors and personal equation.

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[10] "Effect of Time of Mixing on the Strength of Concrete." *Proceedings* A. C. I., 1918, and *Canadian Engineer*, Aug. 1 and 8, 1918; "Design and Control of Concrete Mixtures." Portland Cement Association, Jan., 1927, fig. 2, p. 6. Also given in earlier edition and Concrete Data for Engineers and Architects. Bull. 10 (1921), p. 20, Port. Cem. Assoc.-Lewis Inst.

[11] Water Gain and Allied Phenomena, *Engineering News-Record*, Feb. 10, 1927.

Everyone of the systematic factors can be plotted against strength (as a measure of quality) and a definite relationship established for any particular mixture or range of conditions. This has been very thoroughly done for (b) and (c) and much less comprehensively for (d), (e) and (g).

In general none of the *chance factors* will respond to plotted or analytical treatment as regards relationship to strength or quality. Certain elements of (f) (as size, fineness modulus, quantity, etc.) could be conceivably so handled but since there is not as yet agreement on these points, factor (f) will best remain in the chance list for those who do not prefer to exclude it altogether. By limiting the range of an investigation, or project to one cement, a constant aggregate and uniform manipulation, one can control the influence of *chance factors* upon the resulting quality, or by varying but one of them the effect within the range of variation can be investigated and perhaps evaluated.

The cases in which the *chance factors* can be so limited are always restricted and special. In the general case the effect of these factors will always be present as a disturbing influence of magnitude and importance. Thus, to refer to the systematic factor of which we know the most and which will therefore serve best as a common denominator of expression, the ratio of mixing water to cement, we note that:

1. *Different laboratories have obtained widely different equations or curves expressing the relationship between compressive strength and water-cement ratio.* Fig. 1 (A, B and E) show maximum spreads of 1,755 lb. per sq. in. and 113 per cent strength range at given water-cement ratios. The percentages are in terms of the strengths given by the Abrams upper curve

$$S = \frac{14{,}000}{7^x}.$$

2. *The same laboratory will derive widely different w/c curves in different series of tests.* Data from the outstanding concrete laboratory of the country, the Portland Cement Association's published results, are especially interesting in this connection (Fig. 6, A, B and E). On the basis of the several curves that are plotted, representing large numbers of tests extending over a period of about ten years the spreads reach maximum values of 2,290 lb. per sq. in. and 108.2 per cent of the Abrams upper curve respectively. On the other hand if only the Am. Soc. C. E. series of about 1925 be considered the maximum spreads are found to be 1,610 lb. per sq. in. and 80.9 per cent. Excluding the very low minimum value at 7.5 gal. per bag and using the next lowest value as the minimum these become 1,190 lb. per sq. in. and 52.2 per cent respectively. It is to be assumed that cement, kind of aggregate, and manipulation were held constant in addition to all the factors designated as "systematic." Proportions and grading of aggregate were varied since the purpose of the Am. Soc. C. E. series was to demonstrate the adequacy of the water-cement-strength relation, i. e., to prove that variations in quantity and grading of aggregates are negligible in strength effect for workable mixtures. Whether or

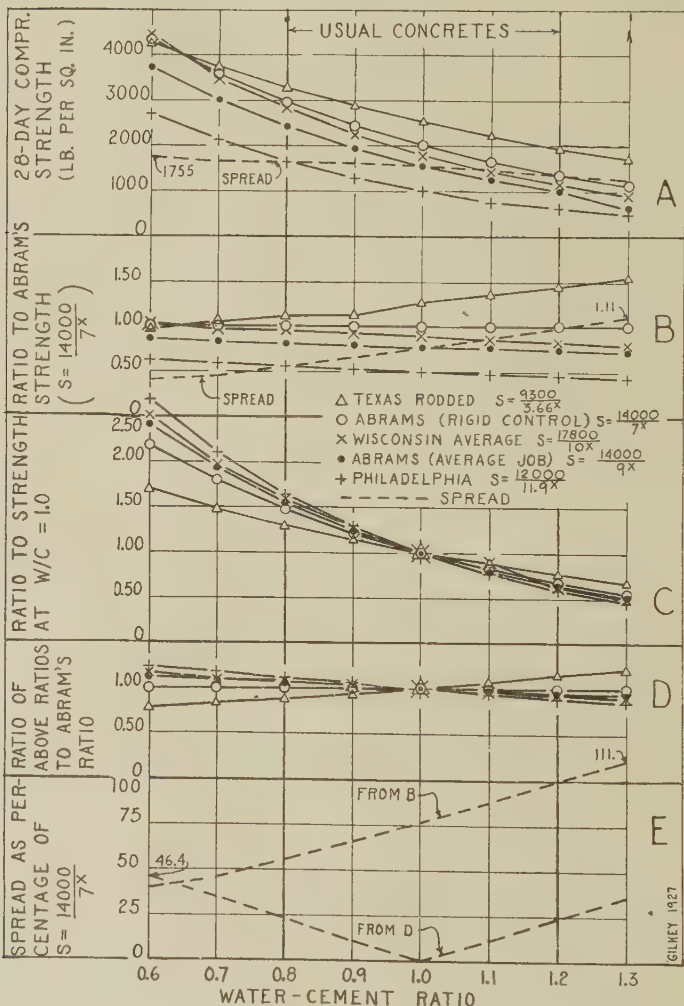

FIG. 1 EQUATIONS OF DIFFERENT LABORATORIES COMPARED

Relation between water-cement ratio and strength. (See Proc. A. S. T. M. 1927 Report of Committee C9 Appendix I) Detailed explanations of curves elswhere in the paper.

not this has been satisfactorily proved will depend upon the individual's interpretation of the evidence presented. The writer, for example, cannot feel that the utopia of attainment has been reached while the spread is still as high as 1,610 or 1,190 lb. per sq. in. and 80.9 or 52.2 per cent respectively, based upon Abrams' upper curve or 57.1 and 36.5 per cent based upon the average strengths for the Am. Soc. C. E. series (a fairer basis for the comparison that applies only to that series). It should be remembered that each maximum and minimum value is already an average

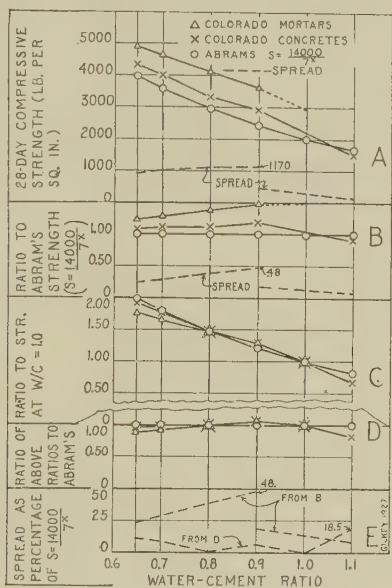

FIG. 2 COLORADO TESTS - MISCELLANEOUS

W/C curves for mortars and concretes. (See Proc. A.C.I 1927 Fig 7 p. 376). Various proportions. Concretes 6 by 12 in. cyl. Mortars 6 by 12 in. and 2 by 4 in. Dotted portions are extensions. See detailed explanations elsewhere in paper.

from five specimens carefully made and tested on different days. Even the abnormally low point at water content of 7.5 gal. per bag can hardly be considered in the light of one individually erratic result, for it too is the average of five separate tests. It would be interesting to see the curves for the individually plotted specimens. Even after obviously erratic specimens were excluded there is no question about the greatly increased spread that would be present. It is stated that no allowance for the effect of absorption upon the water-cement ratio was made. The possible influence of this factor is not only small in comparison with the spread but there is no apparent indication that an absorption correction would consistently

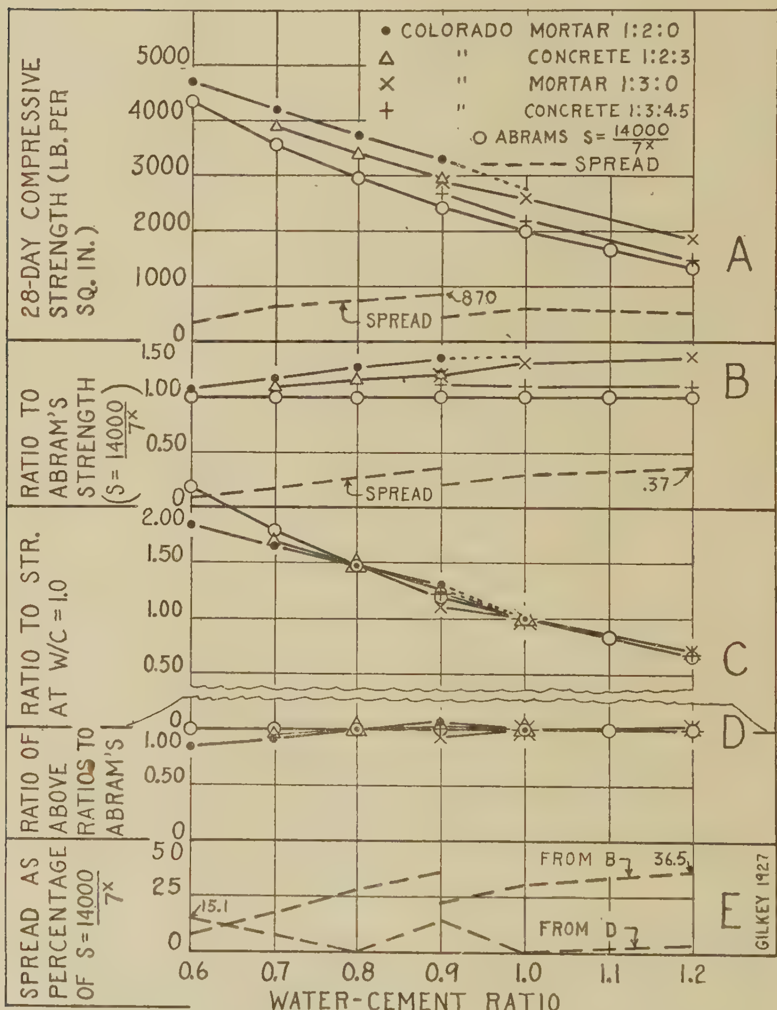

FIG. 3 COLORADO TESTS SPRING 1927

Comparison of mortars and concretes. All specimens 6 by 12 in. cyl. Cement and aggregates constant. Dotted portions are extensions. See detailed explanations elsewhere in paper.

diminish the spread. It would increase it in some cases and lessen it in others.

The remaining figures emphasize the same point on the basis of evidence from other representative sources.

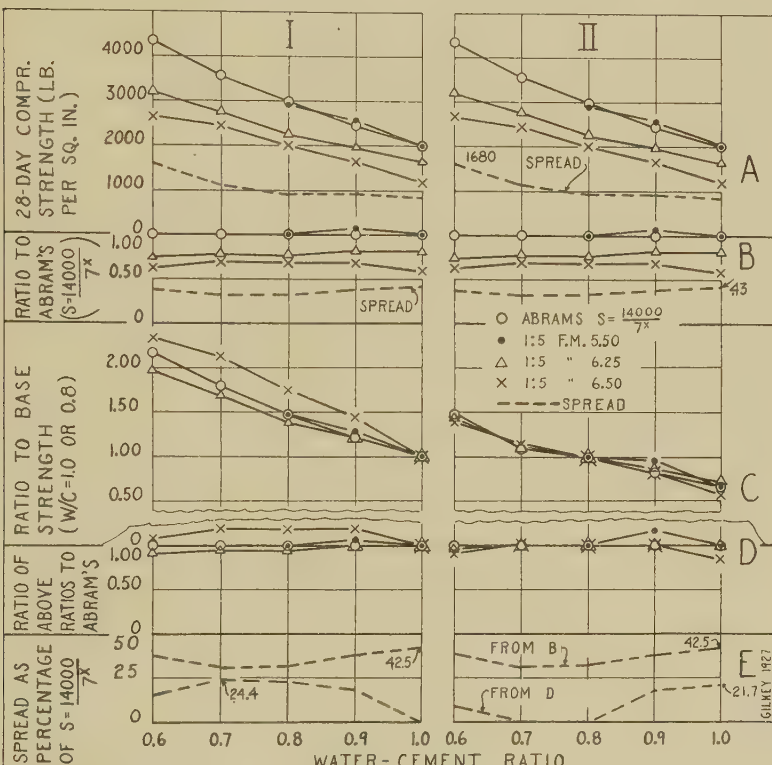

FIG. 4 WISCONSIN TESTS

(R.A.Nelson, Appendix I Report of Comm. C9 A.S.T.M. Proc. 1927 Table 1 For lean mixtures of same series see Fig. 8.).
I Comparison on basis of strength at W/C = 1.0
II " " " " " " W/C = 0.8
See detailed explanations elsewhere in paper.

The purpose of the foregoing discussion is to clarify present status. What is true regarding the limitations of water-cement ratio for actual strength prediction, is doubtless true for each of the other *Systematic Factors.* In attempting to make general application of any of them, the *Chance Factors* will always be present as disturbing elements. Talbot[12]

[12] *Proceedings,* A. C. I., 1927, p. 406.

emphasizes their importance. The great divergence of rate of strength gain with the period of moist storage is evidence of the same. Experiments on the effect of temperature on strength gain are so meager[13] that neither the *systematic relation* nor the limits of the *chance factor* effects are adequately known.

2. THE METHOD EXPLAINED.

In what is to follow, it is understood that the desire is to predict or anticipate, with the greatest possible certainty, either:

(a) The general quality of the concrete, or

(b) Some specific property needed for the work at hand.

Any appropriate property might be measured as an index to either general quality or specific property. The commonest one in the past has been the compressive ultimate strength but it need not be. Flexural strength, surface hardness, tensile strength, watertightness, etc., might be selected as an appropriate quality or property index. To avoid repetition the term strength will be used to indicate whatever property might be meant.

From the store of accumulated experimental data there have been derived various curves or equations expressing the relationship between strength and (a) water-cement ratio, (b) voids-cement ratio, (c) period of moist curing, (d) curing temperatures or (e) time of mixing.

As has been mentioned such relationships have been established with varying degrees of thoroughness. If any of these relationships has been found to be reasonably absolute and invariable over a sufficiently wide range of mixtures, cements, and incidental factors (as has sometimes been claimed for some of them) no further manipulation need be undertaken than to consult the established and accepted strength curve for that particular *systematic factor*. Frequently the curves will be found to agree with one another quite well in general trend but will differ considerably in the actual strength values indicated for given conditions. It is to this situation that the present treament applies, i. e., to relations not identical but reasonably parallel.

Supposing the strengths to be plotted vertically, then if the curves for a given factor be moved upward or downward until any desired pair of points is in contact, the agreement is perfect at that point and the divergence to the right or left of the contact point is solely a function of the non-parallelism that may be present. If the relationships were perfectly parallel, the curves would, of course, coincide.

Select a curve which is deemed, for example, to be representative of the shape or form of the Strength vs. Age relationship. Compute the

[13] Practically all quoted evidence on this subject comes from the experiments of A. B. McDaniel, Bulletin 81, University of Illinois Engineering Experiment Station. These tests were performed in 1914-5. In the light of the great changes in materials and technique since that time there is no more urgent need in the realm of concrete than an adequate investigation of curing vs. temperature.

ordinates as percentages or ratios of the ordinate at 28 days, 3 months, 5 years or any other desired basic age. With this established standard curve or table as a basis, the probable strength at any age of moist curing (other factors constant) can be predicted from the results of tests on

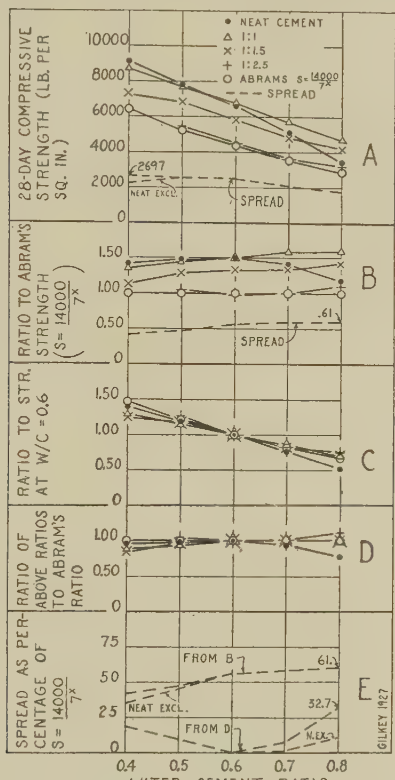

FIG. 5 STANDARD SAND MORTARS (COLO.)

Dec. 1926 series. Proportions by weight. All specimens 2 by 4 in. cyl. (See also Fig. 7 for leaner mixtures of same series). Note that these are compared on the basis of W/C=0.6 See detailed explanation elsewhere in paper.

specimens from the cement, materials, etc., tested at a single age. The error present will be that due to lack of parallelism between the actual and assumed relationships. In the case of some factors (one of which may be the age factor) there is no one curve, whose shape is near enough to all, to permit of any great accuracy in the prediction *but the prediction on the basis of ratios or percentages will, in general, be more accurate than a similar one on the basis of actual strengths.* The portion of the spread due

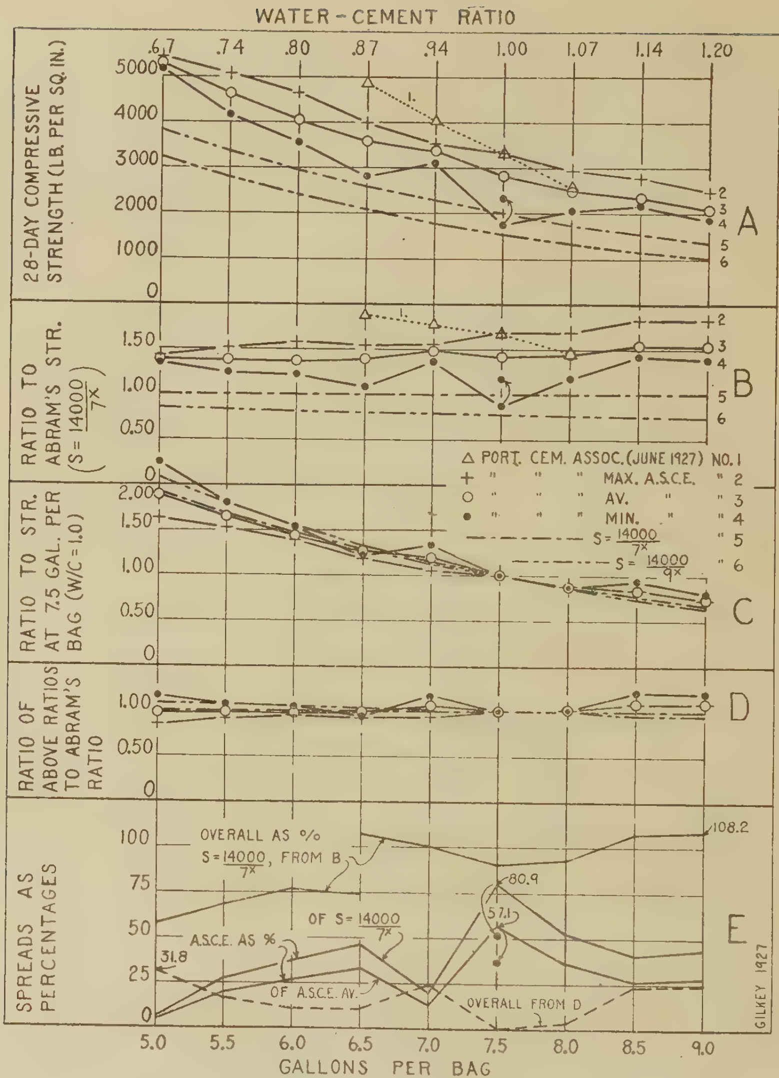

FIG. 6 PORTLAND CEMENT ASSOCIATION TESTS

Published P.C.A. Tests from the following sources:-

1. June 1927 P.C.A. Leaflet "T-116-60M-6-27-9" also "Concrete" Aug. 1927 p. 31.
2. A.S.C.E. Max. } *Trans. A.S.C.E. Vol. 91 (1927) Table I. p. 357*
3. " Av. } *Journal Western Soc. Engrs. Jan. 1927 p. 30.*
4. " Min. } *Eng'r'g News Record Mar. 17, 1927 p. 445.*

Each plotted point on the Max. & Min curves (2) & (4) represents 5 tests on different days. Each point on curve 3 (Av.) represents mean for 6 to 17 different proportions of mortar and concrete i.e. 30 to 85 individual test results. Cement, kind and grading of aggr., constant Proportions varied- (P.C.A. Series 186)

5. Standard Abrams W/C curve for rigid control $s = 14000/7^x$
6 Abram's curve for average job concrete $s = 14000/9^x$
Curves 1-6 incl. represent recent tests.

__Note__. Curve 4 of C was computed on the basis of 2320 (next to lowest tabular strength) instead of 1740 which is abnormally low. Both points are plotted.

to the vertical displacement of the curves has been eliminated and only that due to lack of parallelism remains.

The Strength vs. Water-Cement ratio is the one relation upon which adequate data are available for giving the system a fair workout.

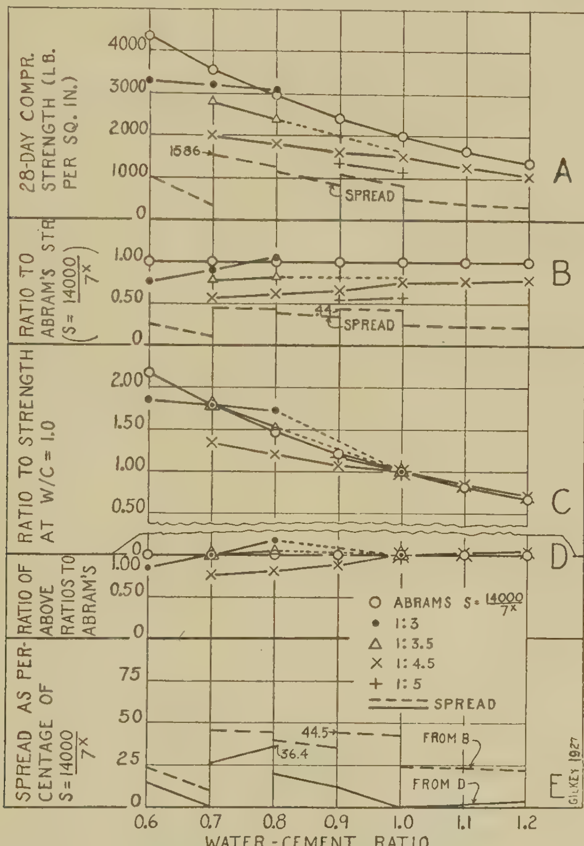

FIG. 7 LEAN MIXTURES OF STANDARD SAND MORTARS COLORADO DEC. 1926 SERIES

Proportions by weight. All specimens 2 by 4 in. cyl. (For others of same series see Fig. 5). Dotted portions are extensions. See detailed explanations elsewhere in the paper.

For that reason and also because the purpose of the present paper is to expound a method rather than to exhaustively apply it, the Strength vs. Water-Cement ratio has been selected for illustrative treatment and all graphical matter shown applies to it. Identical treatment can be accorded

each of the other four factors listed as *systematic.* There are some data available for such treatment in each instance (see Tables 1-5 incl.) but the factors will doubtless require considerable modification from time to time as more complete evidence is accumulated. Moreover there remains much

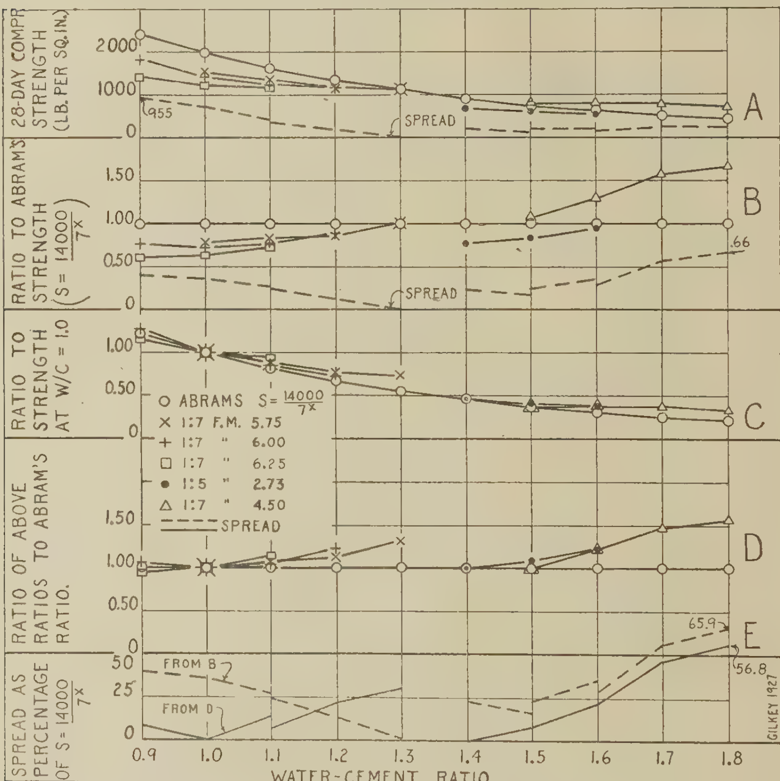

FIG. 8 WISCONSIN TESTS

Lean mixtures covering a wide range of Fineness Modulus. (R.A. Nelson, Appendix I Report of Comm. C9. A.S.T.M. Proc. 1927. Table I). Illustrating a variety of gradings that give W/C curves so far from parallel that the recommended treatment fails to improve the agreement for any considerable W/C range. Only the portions of the W/C curves lying to the right of maximum strength (i.e. the workable range) are plotted. For others of the same series, see Fig. 4.

uncertainty as to whether any usable degree of parallelism exists for such factors as Strength vs. Age, Curing Temperature and Time of Mixing. The factors of Tables 1, 4 and 5 (although probably representative of the best ones available), are essentially unchecked in that their general applicability and probable spreads are not established. Judging from the great

variability in rate of strength gain with age of moist curing that was indicated by the Colorado results of the co-operative tests on thirty-two brands of cement by fifty-two laboratories[14] the prospect of finding a good average Strength vs. Age Curve appears to be remote. In spite of the above mentioned discouraging evidence, the ratios of Table 4 do seem to apply rather well to a wide range of usual mixtures.

There is no question about close parallelism of the different water-cement ratio curves, and the virtue of the method over most of its range (Fig. 1-6). The advantages are not great for very weak mixtures (lean or wet) as is shown by Figs. 7 and 8. The method, as regards water-cement ratio especially, is not new. Lord,[15] Ahlers[16] and doubtless numerous others have accomplished the same thing in but slightly different fashion. The assumption involved is identical, viz., that of parallelism to Abrams' curve.

Even Abrams and the Portland Cement Association have employed slightly similar methods in much of their literature. Plots of Percentage Strength against Normal Consistency or against the water expressed as the percentage of that producing maximum strength, have often been circulated. Of late, the Water-Cement Ratio Curves appear with Strengths plotted against Water-Cement Ratio by loose volume or as Gallons of Water per Bag of Cement.

There are several good reasons why this is probably preferable to the former occasional treatment.

1. Normal consistency is an elusive term not as widely understood as it might be.

2. Maximum strength is extremely variable because maximum strength mixtures are very stiff and a concrete giving maximum strength for one operator or condition of placing would prove to be a weak honeycombed product in the hands of another. Such an uncertain territory is a poor one to use as a reference datum.

3. Everyone, including the writer, likes definiteness. The writer much prefers the strength vs. w/c curve to any other, providing the word "*approximate*," "*probable*," or "*estimated*" be prefaced to strength. With this modification, the maximum of useful information has been conveyed without overstatement or misleading inference. Moreover, in the absence of test results to tie the particular job mixture to the curve, there is

[14] See Report of Committee C-1 (Cement) *Proceedings*, A. S. T. M., 1927, for reports from other laboratories.

[15] A. R. Lord, *Proceedings*, A. C. I., 1927, p. 40; shows several Abrams' curves plotted, using different values for the constant in the denominator. Finds suitable equation for strength of that particular job concrete by interpolation of plotted job test results between the different curves.

[16] J. G. Ahlers, *Proceedings*, A. C. I., 1926, pp. 165 and 175; also *Proceedings*, A. S. T. M., 1927; discussion on field control of the quality of concrete. He says: "The procedure followed by our contracting organization has therefore resolved itself into establishing so-called 'Job Curves' for every operation. When one or two points are found on this curve, it will always be parallel to the water ratio curve and the desired water-cement ratio for the strength wanted in the structure can be arrived at graphically and mechanically without computations by the use of a diagram which is the basic principle of a mechanical device used throughout our concrete operations."

available an estimated strength to be accepted until something better is found.

The Relative Consistency, etc., of Abrams or the P. C. A. resembles the methods later developed and used by Lord, Ahlers, and the writer (as well as probably many others of which there is no printed record), the essential difference being that Abrams was basing his comparison upon one end (the more uncertain one at that) whereas the others are matching curves in the midst of the workable range.

The only originality (if any there be) in the present treatment is the extension to other *systematic factors* of a principle or device already successfully applied to the Strength vs. Water-Cement Ratio Relation. And as previously stated this must be adopted with an open eye, a questioning mind and a considerable exercise of engineering judgment. It can at least be said that here is offered the semblance of a rational approach to territory that was heretofore entirely out of reach for most persons. Thoughtful consideration from even a questionable premise should be preferable to blind indifference. Figs. 1 to 6 show, quantitatively, what can be done with the best known of the *systematic factors*. Probably as much could be accomplished in diminishing the spread for the Voids-Cement Relation. Because of its limited use, at present, and also because of similarity to the Water-Cement Ratio in status, neither plots not tabular matter, regarding it, are offered.

Easily applied constants based upon the best available information are offered in Tables 1 to 5. These cover Time of Mixing, Water-Cement Ratio, Age, and Temperature.

TABLE 1.—TIME OF MIXING VS. COMPRESSIVE STRENGTH (RATIOS).

NOTE.—Data compiled from Portland Cement Association Tests (1917–1918).
All strengths shown as ratio to that for full 1 mm. mix at same age.
Never mix less than 1 mm.
Little gain from extra mixing time at ages over 28 days.
Greatest value of extra mixing is for early strength concrete.
Speeding up mixer beyond designed speed does not help the mixing process. Centrifugal force tends to hold mass near rim of mixer drum.
A wet concrete is not likely to respond to extra mixing as much as will a stiff mixture. (Slater and Walker, *Proceedings* A. S. C. E., 1925, p. 5.)

Line	Column No. 1	2	3	4	5	6
	Mixing Time in Minutes	1	1½	2	5	10
1	3[1]-Day Strength	1.00	1.20	1.40	2.00	2.50
2	7-Day Strength	1.00	1.15	1.30	1.72	1.93
3	14-Day Strength	1.00	1.08	1.16	1.50	1.70
4	28-Day Strength	1.00	1.06	1.14	1.29	1.37
5	3-Month Strength	1.00	1.06	1.11	1.23	1.26
6	1-Year Strength	1.00	1.04	1.07	1.17	1.17

[1] No experimental evidence on effect of Time of Mixing on 3-Day Strength. Values obtained by exterpolation. Some indication that effect of extra mixing is even greater at 3 days.

Example:

Given: Concrete to be placed in service at 7 days.

Desired: To know how much the 7-day strength can probably be raised by extra mixing.

Solution: Line 2, Col. 6. A 10-min. mix will give nearly twice the strength of a 1-min. mix (1.93 in Table line 2, col. 6).

TABLE 2.—WATER-CEMENT RATIO VS. COMPRESSIVE STRENGTH (RATIOS)

NOTE.—Ratios are all on basis of *Abram's Equation* for *Rigid Control:* $S = \frac{14{,}000.}{7^x}$ Strengths of line 14 are those corresponding to this curve and those of line 15 correspond to *Abram's Average Job Equation* $S = \frac{14\,000.}{9^x}$ Use estimated strengths of lines 14 and 15 only in case test results from similar concrete are not available. These ratios will give very accurate predictions if used for estimating strengths of mixtures having a W/C within 20 per cent either way from that of the trial mix.

Line	Column No.	2	3	4	5	6	7	8	9	10	11	12	13
	W/C =	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20	1.30	1.40	1.50	1.60
1	0.50	**1.00**	0.82	0.68	0.56	0.46	0.38	0.31	0.26	0.21	0.17	0.14	0.12
2	0.60	1.22	**1.00**	0.82	0.68	0.56	0.46	0.38	0.31	0.26	0.21	0.17	0.14
3	0.70	1.48	1.22	**1.00**	0.82	0.68	0.56	0.46	0.38	0.31	0.26	0.21	0.17
4	0.80	1.79	1.48	1.22	**1.00**	0.82	0.68	0.56	0.46	0.38	0.31	0.26	0.21
5	0.90	2.18	1.79	1.48	1.22	**1.00**	0.82	0.68	0.56	0.46	0.38	0.31	0.26
6	**1.00**	**2.64**	**2.18**	**1.79**	**1.48**	**1.22**	**1.00**	**0.82**	**0.68**	**0.56**	**0.46**	**0.38**	**0.31**
7	1.10	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**	0.82	0.68	0.56	0.46	0.38
8	1.20	3.89	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**	0.82	0.68	0.56	0.46
9	1.30	4.73	3.89	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**	0.82	0.68	0.56
10	1.40	5.75	4.73	3.89	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**	0.82	0.68
11	1.50	6.98	5.75	4.73	3.89	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**	0.82
12	1.60	8.53	6.98	5.75	4.73	3.89	3.21	2.64	2.18	1.79	1.48	1.22	**1.00**
13	Gal/Bag..	3.74	4.49	5.24	5.98	6.73	7.48	8.23	8.98	9.72	10.47	11.22	11.97
14	Rigid Control.....	5290	4360	3590	2950	2430	2000	1650	1360	1120	920	760	620
15	Average Job.....	4670	3750	3010	2410	1940	1560	1250	1000	810	650	520	420

Example (*a*)

Given: Strength of 1960 for $W/C = 1.10$ (by test).
Desired: Probable Strength at $W/C = 1.30$.
Solution: Line 7, Col. 10 select ratio 0.68. Probable Strength = 0.68 x 1960 = 1330 lb. per sq. in.

Example (*b*)

Given: Strength of 1960 for $W/C = 1.10$ (by test).
Desired: W/C required to obtain 2500-lb. concrete.
Solution: 2500 ÷ 1960 = 1.28. Follow along line 7. By interpolation select W/C = approx. 0.98.

Example (*c*)

Given: $W/C = 1.10$. No test results are available.
Desired: Probable 28-day strength of this concrete.
Solution: Col. 8. Line 14 or Line 15 gives 1650 or 1250 for rigidly controlled or average job concrete, as the case may be.

Example (*d*)

Given: Desired strength of 2500 lb. per sq. in. for average job concrete. No test results available.
Desired: Approx. W/C required. What will this be in gal. per bag?
Solution: Search for 2500 along Line 15. Fine by interpolation W/C = 0.78 approx. =about 5.86 or 5⅞ gal. per bag (from line 13).

Remarks:

Examples (*a*) and (*b*) use the added precision available from a test of a similar mixture. Any Abrams' curve will give the solutions for (*c*) and (*d*). The higher the permissible w/c the leaner the mixture that can be used for a given slump or the thinner the mixture for given proportions. In general it is preferable to use the leaner, stiffer mixture if properly placeable than the richer but thin one, because of the objectionable segregation, water gain,[1] laitance and lack of economy[2] in using less aggregate than the mixture is able to carry. Stiff concrete is good concrete if not too harsh or unworkable for the conditions of placing.[3]

[1] "Water Gain and Allied Phenomena," *Engineering News-Record,* Feb. 10, 1927.
[2] "Recommended Building Code Requirement for Working Stresses in Building Materials." Bureau of Standards BH9 (1926), Table 1, pp. 5 and 6; and Report of Committee C-6, *Proceedings,* A. C. I., 1927, p. 634.
[3] A. R. Lord, *Proceedings* A. C. I., 1927, p. 29.

TABLE 3.—GALLONS PER BAG VS. COMPRESSIVE STRENGTH (RATIOS).

Ratios are all on basis of *Abram's Equation for Rigid Control* $S = \frac{14{,}000}{7^x}$. Strengths of line 15 are those corresponding to this curve and those of line 16 correspond to *Abram's Average Job Equation* $S = \frac{14{,}000}{9^x}$. Use estimated strengths of lines 15 and 16 only in case test results from similar concrete are not available.

Table is identical with Table 2, except that Water-Cement Ratios are expressed as Gal. per Bag (except line 14 of Equiv. *W/C*.).

For illustrative use of Table 3, see examples and remarks under Table 2.

Line	Column No.	2	3	4	5	6	7	8	9	10	11	12	13	14
	Gal/Bag	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0
1	4.0	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40	0.35	0.31	0.27	0.24	0.21
2	4.5	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40	0.35	0.31	0.27	0.24
3	5.0	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40	0.35	0.31	0.27
4	5.5	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40	0.35	0.31
5	6.0	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40	0.35
6	6.5	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46	0.40
7	7.0	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59	0.52	0.46
8	**7.5**	**2.49**	**2.18**	**1.92**	**1.68**	**1.48**	**1.30**	**1.14**	**1.00**	**0.88**	**0.77**	**0.67**	**0.59**	**0.52**
9	8.0	2.83	2.49	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67	0.59
10	8.5	3.23	2.83	2.49	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77	0.67
11	9.0	3.67	3.23	2.83	2.49	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88	0.77
12	9.5	4.19	3.67	3.23	2.83	2.49	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**	0.88
13	10.0	4.76	4.19	3.67	3.23	2.83	2.49	2.18	1.92	1.68	1.48	1.30	1.14	**1.00**
14	*W/C*............	0.53	0.60	0.69	0.74	0.80	0.87	0.94	1.00	1.07	1.14	1.20	1.27	1.34
15	Rigid Control....	4950	4340	3810	3350	2940	2580	2270	1990	1750	1530	1350	1180	1040
16	Average Job......	4320	3730	3220	2780	2400	2070	1790	1550	1330	1150	1000	860	740

TABLE 4.—AGE VS. COMPRESSIVE STRENGTH (RATIOS).

Age means period of continuous moist curing. Concrete is dormant when dry but curing will resume at diminished rate if moisture again becomes available. If concrete is always to be used in air-dry condition 20 per cent may be added to the computed or observed strength when moist. Ratios are derived from a study of large numbers of published test results including those of the Portland Cement Association. They are intended to apply to mixtures having from 4.5 to 9.0 gal. water per bag of cement (*W/C* from 0.60 to 1.20). Although there will doubtless be considerable variation for different brands of cement, these ratios are probably more reliable than anything along this line heretofore published. Strengths under 7 days are especially likely to be erratic as are the strengths of mixtures much richer or much leaner than average concretes of the water-cement ratios mentioned. Nevertheless the early age ratios will be especially useful as an aid in determining when forms may be removed or concrete placed in service. Concrete in dry air may be considered stationary[1] so far as strength change is concerned, after the 20 per cent or more added strength, due to drying, has been acquired. (This will be lost again upon immersion.) Lord (A. C. I. 1923, p. 258) gives somewhat lower ratios for early age strengths. Slater's Rule (A. C. I. 1926, p 437) gives ratio of 7 days to 28 days strength of from 42.5 per cent of 1000 lb. concrete to 66.0 per cent for 5000 lb. concrete. For conservative estimating, use ratios below these if early strengths are being predicted from a known or estimated strength at greater age. Follow reverse procedure if a later strength is being predicted from an early test result. Strong mixtures develop a greater percentage of their ultimate strength at early ages, as indicated by Slater's rule.

Line	Column No.	2	3	4	5	6	7	8	9	10	11	12	13
	Age	24 Hours	2 Days	3 Days	7 Days	14 Days	21 Days	28 Days	3 Month	6 Month	1 Year	3 Years	5 Years
1	24 hours	**1.00**	1.80	2.30	3.30	4.10	4.60	5.00	6.40	7.20	8.10	9.40	10.0
2	2 days	0.56	**1.00**	1.28	1.83	2.28	2.56	2.78	3.56	4.00	4.47	5.22	5.56
3	3 days	0.44	0.78	**1.00**	1.43	1.78	2.00	2.17	2.78	3.13	3.50	4.09	4.35
4	7 days	0.30	0.55	0.70	**1.00**	1.24	1.39	1.52	1.94	2.18	2.44	2.85	3.03
5	14 days	0.24	0.44	0.56	0.80	**1.00**	1.12	1.22	1.56	1.76	1.96	2.29	2.44
6	21 days	0.22	0.39	0.50	0.72	0.89	**1.00**	1.09	1.39	1.57	1.75	2.04	2.17
7	**28 days**	**0.20**	**0.36**	**0.46**	**0.66**	**0.82**	**0.92**	**1.00**	**1.28**	**1.44**	**1.61**	**1.88**	**2.00**
8	3 months	0.16	0.28	0.36	0.52	0.64	0.72	0.78	**1.00**	1.13	1.26	1.47	1.56
9	6 months	0.14	0.25	0.32	0.46	0.57	0.64	0.69	0.89	**1.00**	1.12	1.31	1.39
10	1 year	0.12	0.22	0.29	0.41	0.51	0.57	0.62	0.80	0.89	**1.00**	1.17	1.24
11	3 years	0.11	0.19	0.24	0.35	0.44	0.49	0.53	0.68	0.77	0.86	**1.00**	1.06
12	5 years	0.10	0.18	0.23	0.33	0.41	0.46	0.50	0.64	0.72	0.80	0.94	**1.00**

[1] Air-cured concrete may gain strength slowly by extracting some moisture for hydration from the air. Moist cured concrete exposed to dry air may lose strength slowly due, perhaps, to a slight loss of moisture from some of the crystals. Both of these effects may be generally ignored.

Example (*a*)

Given: Std. 7 day strength 1500 lb. per sq. in.
Desired: To know probable strength at 14 days (1) (moist condition); (2) 14 days plus 2 weeks air-drying.
Solution: Line 4, Col. 6, gives ratio 1.24.
$1.24 \times 1500 = 1860$ (moist).
$1.20 \times 1860 = 2230$ (air-dried).

Remarks:

A few hours immersion will saturate concrete to the moist condition. Several weeks of drying in air may be required to gain air-dry strength. As curing continues until drying halts it, the separate effects are difficult to evaluate at early ages. Dryness of air, volume-area ratio of concrete, its density, etc., all influence rate of drying. (See *Proceedings* A. C. I. 1926, pp. 395–436 [especially 424–431] and *Transactions* A. S C. E., Vol. 91 pp. 153–167).

TABLE 5.—CURING TEMPERATURE VS. COMPRESSIVE STRENGTH (RATIOS).

The table represents the generalized results of the A. B. McDaniel Tests. Bulletin 81, University of Illinois Engineering Experiment Station. These tests were performed in 1913–14. Reliable data on curing temperature effects are extremely meager Moreover a considerable exercise of judgment is necessary in deciding just what the curing temperature shall be assumed to be in a particular case. Heating of aggregates, exposure to cold winds, chemical heat generated in setting, etc., all have their bearing on the problem. Lord (Notes on Wacker Drive, *Proceedings* A. C. I. 1927) shows results nearly identical with these. The ratios vary little for different test ages. If the concrete becomes dry, curing ceases regardless of temperature. This point is especially important in winter concreting. The artificial heat applied may halt curing due to lack of moisture.

Line	Column No.	2	3	4	5	6	7	8
	Degrees F.	30	40	50	60	70	80	90
1	30	**1.00**	1.36	1.66	1.91	2.13	2.28	2.43
2	40	0.73	**1.00**	1.22	1.41	1.56	1.67	1.78
3	50	0.60	0.82	**1.00**	1.15	1.28	1.37	1.46
4	60	0.52	0.71	0.87	**1.00**	1.11	1.19	1.27
5	70	**0.47**	**0.64**	**0.78**	**0.90**	**1.00**	**1.07**	**1.14**
6	80	0.44	0.60	0.73	0.84	0.93	**1.00**	1.07
7	90	0.41	0.56	0.68	0.79	0.88	0.94	**1.00**

Example:

Given: Specimens cured at average temperature of 70 deg. gave standard 28-day strength of 2400 lb. per sq. in.
Desired: 28-day strength at 40 deg. F.
Solution: Col. 3, line 5 select ratio 0.64.
$0.64 \times 2400 = 1540$.

COMBINING SIMULTANEOUS FACTORS. ILLUSTRATIVE EXAMPLES USING TABLES 1-5.

NOTE. For the use of the tables individually, i. e., for one variable only, see illustrative examples given in connection with each table. The following examples deal only with the rational analysis for several variables simultaneously present.

EXAMPLE (a)

Given: Test specimens from given materials. $w/c = 0.90$; strength at 7 days (moist cured moist tested) $= 1{,}930$ lb. per sq. in.; curing temperature was 60 deg. F.; mixing time was 2 min.

Desired: Probable 14-day strength of concrete from same materials, mixed 1 min.; w/c of 1.10; curing temperature 40 deg.; moist cured for 7 days but location, mass of concrete, and exposure such that it seems safe to assume that curing was able to continue without serious interruption through the tenth day.

Solution:

28-day std. strength same *w/c*, temperature, etc.
(Table 4, Line 4, Col. 8.) $1.52 \times 1{,}930 = 2{,}940$

28-day std. strength at $w/c = 1.10$
(Table 2, Line 5, Col. 8.) $0.68 \times 2{,}940 = 2{,}000$
(Using about 8¼ gal, per bag—Line 13, Col. 8.)

28-day std. strength at temperature 40 deg. F.
(Table 5, Line 4, Col. 3.) $0.71 \times 2{,}000 = 1{,}420$

28-day std. strength at 1 min. mix
(Table 1, Line 2, Cols. 2 and 4.) $\dfrac{1.00}{1.18}\, 1{,}420 = 1{,}200$

14-day strength of above concrete
(Effective age of moist curing 10 days)
(Table 4, Line 7, Cols. 5-6, by interpolation)
Strength $= 0.73 \times 1{,}200 = 870$ lb. per sq. in.

If service conditions are such that there is no possibility of the concrete being subsequently re-saturated while in service, use can be made of the drying strength acquired after curing strength-gain ceased. It is impossible to evaluate the effect of the 4-day drying but safe to assume that it is the equivalent of what 4 days more of moist curing would have given. On this basis (from Table 4, Line 7, Col. 6.) Strength $= 0.82 \times 1{,}200 = 985$ lb. per sq. in.

Under similar conditions, after the concrete had air dried to maximum and practically constant strength (say at 21 days or older), the 20 per cent drying strength allowance could be added to the 870 lb. per sq. in. moist strength at 10 days, i. e.,

Strength $= 1.20 \times 870 = 1{,}040$ lb. per sq. in.

Summarizing, this concrete could be expected to carry 870 lb. per sq. in. moist; 985 lb. partially dry as at 14 days; 1,040 lb. eventually if kept dry. If subsequently wet the strength would at once drop to 870, then gradually start to build up permanent additional strength by resumed curing, i. e., added hydration.

It should be understood that the 20 per cent as the added allowable strength for dry concrete over wet, is about the minimum difference found. It may be as high as 40 per cent, which is in this case added factor of safety. The status is reversed when the strength of moist concrete is being estimated from test results on air-dry cores or specimens. This is one of the most glaring of current fallacies.[17]

Note that the actual solution can be performed as a process of continued multiplication of factors, i. e.,

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[17] *Transactions*, A. S. C. E., Vol. 91 (1927), pp. 153-167; *Proceedings*, A. S. T. M., 1927; discussion by H. J. Gilkey on "The Field Testing of Concrete."

$$1.52 \times 0.68 \times 0.71 \times \frac{1.00}{1.18} \times 0.73 = 0.455$$

$$0.455 \times 1{,}930 = 870$$

It will be noticed that any one line of any of the tables except Table 1, is adequate. For this reason certain lines have been emphasized as suggested ones to apply in using the tables on a one-line basis. These standardized lines are:

$w/c = 1.00$	Table 2, Line 6
$w/c = 7.5$ gal. per bag	Table 3, Line 8
Age = 28 days	Table 4, Line 7
Temperature = 70 deg. F.	Table 5, Line 5

Example (*a*) in tabular form will be given showing both types of solution:

Data	Time of Mixing	*W/C*	Age Moist Cured	Curing Temperature	Product Factors	Strength
Concrete A..................	2 minutes	0.90	7	60 deg. F.	....	1930
Concrete B..................	1 minute	1.10	7 + 3 = 10	40 deg. F.	....	Required

SOLUTION AS GIVEN

Table......................	1	2	4	5	....	....
Line......................	2 and 3	5	4	4	....	....
Column......................	4 and 1	8	{ Between 5 and 6 }	3	....	....
Factor......................	1/1.18	0.68	1.10	0.71	0.45	....
				0.45 × 1930		= 870

SOLUTION—ONE LINE METHOD

Table......................	1	2	4	5	....	....
Line......................	2 and 3	6	7	5	....	....
Columns......................	4 and 1	8 ÷ 6	(5 to 6) ÷ 5	3 ÷ 5	....	....
Values......................	1/1.18	.82/1.22	.73/.66	.64/.90	....	....
Factor......................	0.85	0.68	1.10	0.71	0.45	....
				0.45 × 1930		= 870

NOTE.—The emphasized lines of the tables can advantageously be plotted for ease of use.

EXAMPLE (b)

Same data as Example (a) but is desired to know what period of moist curing should be employed in order to insure a strength of 1,100 lb. per sq. in. for the concrete in the moist condition.

Solution: Use same factors as for (a) except the last one which is unknown, i. e.,

$$1.52 \times 0.68 \times 0.71 \times \frac{1.00}{1.18} \times C \times 1{,}930 = 1{,}100$$

$$C = \frac{1{,}100}{0.62 \times 1{,}930} = 0.92$$

Table 4, Line 7, Col. 7, gives 21 days as the moist curing period requirement.

With equal simplicity the possibility of altering the curing temperature, water-cement ratio, or time of mixing could be separately investigated and compared.

EXAMPLE (c)

Suppose that no test results are available from the cement and materials at hand. What is probable 14-day strength under same conditions as Example (a)?

Solution: Abrams' estimated 28-day strength for $w/c = 1.10$ will be the starting point. Let us assume the case of rigid control.

Table 2, Line 14, Col. 8, or any Abrams' curve gives $S_{28} = 1{,}650$ lb. per sq. in. This is probable equivalent of 1 min. mix wet cured at 70 deg. F. and moist at test.

Table 5, Line 5, Col. 3. Temperature Factor $= 0.64$

Table 4, Line 7, Cols. 5-6. Age Factor $= 0.73$

(Assuming that moist curing continued for 10 days)

Estimated moist strength $= 0.64 \times 0.73 \times 1{,}650 = 820$ lb. per sq. in.

The foregoing illustrative problems should serve to suggest the numerous possible uses to which the method may be put. The method, as such, is direct and definite. Its uncertainties lie in the values and invariableness of the constants selected. These can be replaced by better ones as found. By no stretch of the imagination can the method be construed as likely to furnish results less accurate than values that are not now even guessed at. The few who have tried to intelligently gage the effects of others of the *systematic factors* than the one or two at present most in the spotlight have usually found themselves able to tap certain pioneer literature on the subject without tangible means for applying it to the specific problem. It is hoped that the method outlined may serve, not as the last word in either correctness of constants, or perfection of method, but rather as a much needed start toward rational solutions for such problems.

4. EXPLANATIONS OF THE FIGURES.

The figures are somewhat in the nature of an appendix to the paper. A study of them will enable one to judge for himself whether the Water-Cement Ratio vs. Strength is to be accepted as absolute or relative. It is also possible for him to differentiate between that portion of the spread that is attributable to lack of parallelism of different w/c curves and that which is due to bodily displacement. In other words the evidence from

which the method of Ahlers,[15] and Lord[16] and others can be judged is here presented. With a similar mass of authoritative data, the same analysis could be extended to each of the *systematic factors* that have been considered. Some such analysis is needed to test the treatment for probable spreads and to effect their separation into bodily displacement and non-parallelism. As the ratio, or percentage method, will eliminate the error of the former but not of the latter, it is only by such separate diagnosis that a measure of dependability can be obtained for the method in each of its applications.

All of the figures are alike in layout and one general explanation will be given followed by a few remarks on individual figures.

Curves A are the well-known Water-Cement Ratio vs. Strength curves. Abrams' equation for rigidly controlled concrete, $S = \dfrac{14{,}000}{7^x}$ appears on each of them as a uniform standard of comparison. Abrams' other curve $S = \dfrac{14{,}000}{9^x}$ appears on Figs. 1 and 6. On the A curve of all except Fig. 6 is also plotted the *spread*, in lb. per sq. in., as a broken line. In comparing mortars and concretes, or other rich vs. lean, or dry vs. wet mixtures, the experimentally determined curves will only overlap for part of their length. Sometimes the point selected for superposition (usually at $w/c = 1$ in figures) falls beyond one or more of the curves. In these cases an imaginary strength is computed from the ratios of Table 2 or 3 and the extension of the curve is dotted in, as occurs in Figs. 2, 3 and 7. These extensions are parallel to the Abrams' curve by virtue of the constants from which they were computed. Such extensions are not included in the spreads as they do not represent experimentally-determined strengths.

On B, the standard of comparison, $S = \dfrac{14{,}000}{7^x}$, appears as a straight horizontal line at ordinate 1.00. If perfect parallelism existed between the different curves, they too would appear as parallel horizontal lines. The spread, expressed as the ratio of the Abrams' strength, appears as a broken line on all but Fig. 6B.

In C, the curves are superimposed at some convenient w/c value. This may be any value but will best be one about the middle of the w/c range being considered. For concretes superposition at $w/c = 1.00$ is often good. This was used most often in the figures. Fig. 4 shows comparative effects of two different matching values for w/c. Parallel curves would coincide in C and D. The departure therefore represents the degree of non-parallelism present. The strength ratios of C are analogous to those of any hori-

[15] See footnote on page 17.
[16] See footnote on page 17.

zontal line of Tables 2 and 3. Table 6 gives the computations for Fig. 1 which are illustrative.

Table 6 supplies the detail that permits a complete understanding of C, D and E. For hasty study A, B and E are especially recommended as their messages are very easily visualized. The length of ordinate between

TABLE 6.—ILLUSTRATIVE COMPUTATION FOR FIGURE 1.

Line	Column No.	2	3	4	5	6	7	8	9	10
1	Equation	$\frac{14,000}{7^x}$	$\frac{14,000}{9^x}$	$\frac{9,300}{3.66^x}$	$\frac{17,800}{10^x}$	$\frac{12,000}{11.9^x}$	Max.	Min.	Spread	
2	W/C	Abram's Rigid	Abram's Av. Job	Texas Rodded	Wis. Av.	Phila.	....	....	Actual	Per cent Col. 2
3		STRENGTHS (A CURVE)								
4	0.60	4356	3746	4270	4470	2715	4470	2715	1755	40.3
5	1.00	2000	1555	2540	1780	1008	2540	1008	1532	76.6
6	1.30	1115	805	1717	892	479	1717	479	1238	110.9
7		RATIO TO $S = \frac{14,000}{7^x}$ (B CURVES)								
8	0.60	1.00	0.86	0.98	1.02	0.62	1.02	0.62	0.40	40.0
9	1.00	1.00	0.78	1.27	0.89	0.50	1.27	0.50	0.77	77.0
10	1.30	1.00	0.72	1.54	0.80	0.43	1.54	0.43	1.11	111.0
11		RATIO TO STRENGTH AT $W/C = 1.00$ (C CURVES)								
12	0.60	2.18	2.41	1.68	2.51	2.69	2.69	1.68	1.01	46.4
13	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	00.0
14	1.30	0.56	0.52	0.68	0.50	0.48	0.68	0.48	0.20	35.7
15		RATIOS OF LINES 12–14 INCL. COMPARED WITH ABRAM'S RATIOS COL. 1 LINES 12–14								
16	0.60	1.00	1.11	0.77	1.15	1.23	1.23	0.77	0.46	46.0
17	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	00.0
18	1.30	1.00	0.93	1.21	0.90	0.85	1.21	0.85	0.36	36.0

Note that initial spread of E may be obtained from either A or B curves (see check computations, Cols. 9 and 10). Likewise final spread of E can be gotten from either C or D curves. Note similar check values, Cols. 9 and 10, Lines 12–18.

The various ratios can best be visualized by observing which goes to unity in a given operation.

Col. 2, Lines 12–14, agree with Table 2, Line 6.

the two E curves represents the increase in precision that can be gained. Where one mixture is discontinued, there will always be a break in the spread curve unless the discontinued curve lay between the two extremes at the point of stopping.

Additional Comments Upon Individual Figures.—Fig. 1 illustrates the divergence of laboratory results as regards Water-Cement Ratio Law. It also shows the high degree of parallelism that exists, notwithstanding, and which permits the spread to be reduced from 111 to 46.4 per cent, or

within the range of usual concretes, to less than 25 per cent. Of course the matching of curves could be done at any *w/c* and by matching at or near the value of *w/c* to be used the spread can be made to approximate zero. The great strength of the Texas Rodded, especially within the range of the wetter mixtures, is probably accounted for by loss of water from over-tamping. The actual water ratio at time of initial set was probably much lower than what it was supposed to be.

Figs. 2, 3, 5 and 7 are of different Colorado series and should be quite self-explanatory. Due to the large variety of mixtures used, there are many discontinuities. Without exception these figures support the evidence cited a year ago,[18] that "at constant water-cement ratio the leaner mixture is the weaker one."

In like manner the Wisconsin tests (Figs. 4 and 8) supply some evidence to the effect that "at constant water-cement ratio, the concrete made from the larger particles of aggregate is the weaker. There is one exception to this apparent trend in Fig. 8. These are but a small fraction of the Wisconsin series. They were selected more or less at random and without any regard to the point just mentioned. The slight indication may or may not be significant.

Fig. 4, I and II, as mentioned, illustrate the desirability of basing comparisons upon points near the middle rather than near the end of the particular *w/c* range being investigated.

Fig. 6 of the various Portland Cement Association laboratory results has already been rather fully discussed in the early part of the paper.

Figs. 7 and 8 are of lean mixtures in which a slight lack of parallelism introduces very high percentages of spread. Neither of these two lots of data adapt themselves overly well to the treatment since the spread is due more to non-parallelism than to bodily displacement.

Much more could be said relative to the different figures and what they show, but those who are interested can unearth the material at their leisure.

CONCLUSIONS.

It has been rather fully demonstrated that the very useful water-cement ratio method for predicting strengths of concrete can be made much more valuable by means of such procedure or technique as is herein outlined. Similar methods have been successfully applied by others. The figures shown are not a necessary part of the method. They but illustrate its degree of precision for representative cases.

The usual variables of concrete can be divided into two classes:

(a) The *systematic factors.*

(b) The *chance factors.*

If it be assumed that for the (a) class there are reasonably reliable parallel laws governing the behavior of given cements, aggregates and

[18] *Proceedings,* A. C. I., 1927, pp. 363-414.

manipulation, these *systematic factors* are by analogy open to the same sort of treatment and analysis as is the water-cement ratio relation (probably the most important of them). Tables of constants, representing what is believed to be the best of our current knowledge have been compiled and placed in easily usable form. These are supported by typical illustrative solutions.

Knowing the strength that a given cement and aggregate gave under any one set of *systematic conditions*, it should be possible to predict through straightforward methods and with reasonable accuracy the strength that the same cement and aggregates will give for any other known or assumed combination of *systematic factors*. This should be an added step toward co-ordination between the standard laboratory specimen (quality control) and the actual strength of the concrete in the structure. It should produce tangible evidence upon which to base decisions as to time for placing structures into service, actual factors of safety present under conditions of use, etc.

In the absence of a set of test results, Abrams' (or any other) estimated strength may be assumed as a starting point from which to branch out and take rational account of the other Systematic Variables known or assumed to be present.

Acknowledgments.—The writer desires to express appreciation for the encouragement and co-operation of Prof. C. L. Eckel, head of civil engineering. Without such co-operation it would be practically impossible to carry on projects of this sort at an institution where funds are scarce and where there is no provision for organized research. Sartwell Egerton, student in civil engineering, has labored faithfully in reducing data and preparing figures. J. M. Buirgy, senior civil engineering student, has also assisted.

Discussion.—Predicting Concrete Strengths.

Weston S. Evans* and H. Walter Leavitt.*—Prof. Gilkey is to be complimented on his treatment of this most interesting subject. As he states, "There's much that we don't know about concrete and much more that we know but imperfectly or in part." He has taken some of this "imperfectly" known data and applied it to a very interesting problem. A structure can be no stronger than its foundation, and if we build elaborately or otherwise upon data which is fundamentally weak we must expect mediocre results. **Evans and Leavitt.**

The strength of concrete is undeniably affected by such variable factors as age or time of curing, water content, cement content, temperature, etc. We are dealing, then, with a very complicated and intricate maze of variables when we try to guess what the strength of concrete should be. If we have accurate measurements of these major and minor factors which do affect strength, and if we use each of these results in their proportionate relationship we may hope to predict strength with accuracy.

Prof. Gilkey's equation takes the form of

Required strength $= abcde$ (given strength), or $S = abcde(s)$

$a =$ factor to change given strength to 28-day strength, other things being equal.

$b =$ factor to change $a(s)$ to a strength based on the required w/c.

$c =$ factor to change $ab(s)$ to a strength based on the required temperature of curing.

$d =$ factor to change $abc(s)$ to a strength based on a required time of mixing.

$e =$ factor to change $abcd(s)$ to a strength based on a required time of curing.

One difficulty with the method lies in the fact that all factors or ratios are applied to the whole quantity making up the required result. In other words, an error of estimate or an error in knowledge of the temperature of curing will not only affect that part of the required strength controlled by it, but all other parts as well. Let us look at it another way.

Each variable factor contributing to the strength or weakness of concrete has its effect independent of all the others. When the effect of w/c on strength is studied, it is commonly found that one w/c has several given strengths. It has been common practice to average these strengths and thus to secure a smooth curve. This practice is to be deplored, for we

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are throwing away the very weapons with which the solution of our problems is to be met. Was it temperature, was it voids or was it time of mixing or what not that caused this spread? Then why is it not correct to state that the method under discussion is wrong because the whole strength is affected by each variable. The equation to predict strength must be of the form $S = K + ax + by + cz + dq + er$, etc., in which x, y, z, etc., are variables such as w/c, age, temperature of curing, and what else we do not know at present, and a, b, c must be co-efficients derived not by considering each variable in its relation to strength separately but by considering them all combined. Then a slight error in the knowledge of one term will not seriously affect the whole result.

Any such method as stated above could not be applied to the problem as outlined by Mr. Gilkey, with the data at hand. How could we expect ratios controlling temperature from data taken in Colorado to function even approximately with ratios controlling w/c derived from data taken in Illinois at widely different times? From table 5 we gather that the ratios controlling curing temperature are independent of w/c. Let us take an example and see what happens. In Maine we get many specimens mixed for one minute with 7.5 gal. of water per bag of cement that will give a strength of about 2000 lb. when cured at 50 deg. F. and kept wet for 28 days. According to Mr. Gilkey's plan, if these had been stored at 70 deg. they would have tested 2560 lb. and if they had been mixed for 10 min. instead of 1 min. they would have tested 3507 lb. If they had been mixed with 6.5 gal. of water per bag instead of 7.5 gal., they would have tested 4559 lb. and at the age of 5 yr. would have tested 9118 lb. per sq. in. Does this result seem reasonable? Accurate tables and constants cannot be derived in this way. When we make a test specimen we must know its time of mixing and temperature, for the effect of these variables change with the mixture and other things. The problem must be solved in one laboratory and at one time or in closely connected series. If "increased precision" is really desired, why not use sound basic or fundamental data so weighted that all the interrelationships are allowed to function? It would seem that the time is now ripe for the launching of such an investigation.

Notes on the Progress of Some Studies of the Crazing* of Portland Cement Mortars.

By P. H. Bates† and C. H. Jumper.†

Since the paper on crazing[1] was read before the Institute in 1925 it has been possible to inaugurate a program of research on crazing and to make a series of test panels of mortars at the Bureau of Standards. These have now been under observation for various periods and while nothing has been obtained which will definitely settle, or even very strikingly indicate, a satisfactory solution of the problem it is thought that the test procedure and data so far obtained will be of interest. It is particularly hoped, also, that constructive criticism will be obtained.

The several tables will indicate the types of stucco mixes used. The underlying thought of their design was that the sands had a certain surface area and that this area should at least be covered with a layer of cement. Hence they were prepared on the basis of the number of square inches of area on a gram of sand per gram of cement in the mix. To obtain the desired sand area-cement ratio three sands screened between the 4 and 8 mesh, the 8 and 30 mesh and the 30 and 100 mesh, were used. These were mixed in the several proportions as indicated to obtain the other sands used. The several other well-known characteristics of mortars, such as the approximate granularmetric analysis and the fineness modulus, are also presented in the same tables. In all mixes the same cement has been used. This was received in the form of clinker and has been ground from time to time to the same fineness in the Bureau's experimental cement plant.

Three different bases upon which to apply the mortars were first tried in the small preliminary panels (Table 1). These were glass, thin rubber sheeting, and a well-hardened cement mortar. These were to represent an absorptive (mortar base) and a non-absorptive base (glass or rubber), a base forming a maximum (mortar base) and one forming a minimum (glass) adhesive surface, and one serving as a possible maximum (mortar) change in area difference due to drying out of the stucco during hardening, and one as a minimum change in difference (rubber). The desired conditions were obtained except in adhesion, where contrary to

[1] Bates: Crazing on Cement Products: *Proceedings,* American Concrete Institute, Vol. 21, p. 126.

* Publication approved by the Director of the Bureau of Standards of the U. S. Department of Commerce.

† U. S. Bureau of Standards.

expectations the mortar adhered so tenaciously to the glass during the hardening and shinkage that the uncovered surface of the glass was rendered decidedly convex. The data secured from observing these panels are presented in Table 1. The early deportment of these suggested certain additional mortars. These are indicated in Table 1 by either the use of an X or 12. In the fourth column when the former is used it indicates that approximately 10 per cent more water was used than in the first set of mixes indicated by a number not followed by X. When 12 is used it indicates that the ratios of cement to sand in all mixes were such as to give a surface area-cement ratio of from 12 to 14. It will be noted from the table that during from seven to nine months no marked difference was seen in the various panels so far as crazing was concerned. It was therefore thought advisable to change the size of the panels and omit from future work the rubber sheeting base.

A second series was then inaugurated, using as a base either glass plates, 24 x 24 in., or an absorptive mortar base of the same dimensions made of a 1 : 4 dry tamped mortar using a rather coarse concrete sand and some light expanded metal as reinforcing. The same mixes were used as in the first series. The following procedure was employed in making the panels of this series when the mortar base was used. One-half of the base was thoroughly wet down before applying the mortar. The other half was left dry. After applying the mortar one-half of the surface was thoroughly trowelled to bring the finely divided and suspended cement particles to the surface. The other half was given a float finish. Thus, one-fourth of the panel had a floated finish surface on a dry base, another fourth a floated finish surface on a wet base, a third fourth had a trowel finished surface on a dry base and the fourth quarter a trowelled surface on a wet base. This procedure was also followed in the later series presented in Tables 3 to 6. However, these tables do not indicate the condition of the various quarters. This is due to the fact that while generally the trowelled surface on the dry base showed crazing first it spread to the other quarters within a few days and consequently there was no need for indicating in the tables any difference in the appearance of the several quarters.

While these two series were under observation a number of other panels were made under a variety of different conditions. Thus some were subjected immediately after making to the draft produced by an electric fan. Others were wetted down thoroughly at different times after hardening. Others were made on the roof and immediately subjected to whatever atmospheric change took place without protection except from rain. The results which were obtained, but not presented here, led to the conclusion that it would be desirable to start a third series in which panels of the same type would be made under different atmospheric conditions.

The third series (Series M, Table 3), was then made, in which the two mixes previously used containing the coarser aggregates were omitted. Also but one water-cement ratio was used for each mix. Each mix was

Sand		Condition Sept. 6, 1927			Condition Nov. 14, 1927		
Size	Proportion by Weight	Concrete	Glass	Rubber	Concrete	Glass	Rubber
4/8	0.33⅓	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
8/30	0.33⅓	Crazed	"	"	Crazed	"	"
30/100	0.33⅓	lightly crazed	"	"	Slightly crazed	"	"
4/8	0.60	Crazed	Crazed	Crazed	Crazed	Crazed	Crazed
8/30	0.20	"	"	"	"	"	"
30/100	0.20	lightly crazed	"	"	Slightly crazed	"	"
4/8	0.20	Crazed	Crazed	Crazed	Crazed	Crazed	Crazed
8/30	0.60	"	"	"	"	"	"
30/100	0.20	O. K.	"	"	O. K.	"	"
4/8	0.20	Crazed	Crazed	Crazed	Crazed	Crazed	Crazed
8/30	0.20	lightly crazed	"	"	Slightly crazed	"	"
30/100	0.60	" "	"	"	" "	"	"
8/30	0.50	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
30/100	0.50	O. K.	"	"	O. K.	"	"
		lightly crazed	"	"	Slightly crazed	"	"
4/8	0.50	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
30/100	0.50	lightly crazed	"	"	Slightly crazed	"	"
		" "	"	"	" "	"	"
4/8	0.50	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
8/30	0.50	Crazed	"	"	Crazed	"	"
		O. K.	"	"	O. K.	"	"
4/8	1.00	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
		Crazed	"	"	Crazed	"	"
		?	?	?	?	?	?
8/30	1.00	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
		Crazed	"	"	Crazed	"	"
		O. K.	"	"	O. K.	"	"
30/100	1.00	O. K.	Crazed	Crazed	O. K.	Crazed	Crazed
		lightly crazed	Slightly crazed	"	Slightly crazed	Slightly crazed	Slightly crazed
		racked—O.K.	Crazed	"	Cracked—O.K.	Crazed	Crazed

* Hair cracks thro

EXPLANATORY No ar, the sand being graded and proportioned as with Mix I.

Sand	
Size	Proportion by Weight
4/8 8/30 30/100	0.33⅓ .33⅓ .33⅓
4/8 8/30 30/100	.60 .20 .20
4/8 8/30 30/100	.20 .60 .20
4/8 8/30 30/100	.20 .20 .60
8/30 30/100	.50 .50
4/8 30/100	.50 .50
4/8 8/30	.50 .50
4/8	1.00
8/30	1.00
30/100	1.00

See explanatory no

TABLE III.—SERIES M—VARIABLES—GRADATION OF SAND, AND PLACE OF MAKING

Sand		Mortar, Proportion by Weight	Mix Number	Surface Area, sq. in. per gr. cm.	Fineness Modulus	Flow, per cent	Water		Where Made	Date Made	Condition Aug 2, 1927	Condition Sept. 2, 1927	Condition Nov. 1, 1927	Condition Dec. 1, 1927
Size	Proportion by Weight						W/C	Per Cent						
4/8	0.33⅓	1:3	1ML	13.45	3.35	103	0.96	16	Laboratory*	6-27-27	O. K.	Crazed	Crazed	Crazed
8/30	.33⅓	1:3	1MR	13.45	3.35	101	.96	16	Roof†	6-27-27	Crazed	Crazed	Crazed	Crazed
30/100	.33⅓	1:3	1MD	13.45	3.35	103	.96	16	Dark closet*	6-27-27	O. K.	Slightly crazed	Slightly crazed	Crazed
4/8	60	1:3	2ML	9.22	4.00	105	.96	16	Laboratory*	6-28-27	O. K.	Crazed	Crazed	Crazed
8/30	.20	1:3	2MR	9.22	4.00	107	.96	16	Roof†	6-28-27	Crazed	Crazed	Crazed	Crazed
30/100	.20	1:3	2MD	9.22	4.00	108	.96	16	Dark closet*	6-28-27	O. K	Slightly crazed	Slightly crazed	Slightly crazed
4/8	.20	1:3	3ML	11.87	3.52	83	.96	16	Laboratory*	6-28-27	O. K.	Crazed	Crazed	Crazed
8/30	.60	1:3	3MR	11.87	3.52	84	.96	16	Roof†	6-28-27	Crazed	Crazed	Crazed	Crazed
30/100	20	1:3	3MD	11.87	3.52	85	.96	16	Dark closet*	6-28-27	O. K.	Slightly crazed	Slightly crazed	Slightly crazed
4/8	.20	1:3	4ML	19.25	2.71	66	1.02	17	Laboratory*	6-29-27	O. K.	Crazed	Crazed	Crazed
8/30	.20	1:3	4MR	19.25	2.71	69	1.02	17	Roof†	6-29-27	Crazed	Crazed	Badly crazed	Badly crazed
30/100	.60	1:3	4MD	19.25	2.71	68	1.02	17	Dark closet*	6-29-27	O. K.	Crazed	Crazed	Crazed
8/30	50	1:3	5ML	18.70	2.55	73	1.08	18	Laboratory*	7- 1-27	O. K.	Crazed	Crazed	Crazed
30/100	.50	1:3	5MR	18.70	2.55	76	1.08	18	Roof†	7- 1-27	Crazed	Crazed	Crazed	Crazed
		1:3	5MD	18.70	2.55	74	1.08	18	Dark closet*	7- 1-27	O. K.	Slightly crazed	Slightly crazed	Slightly crazed
4/8	.50	1:3	6ML	15.40	3.36	112	1.02	17	Laboratory*	7- 2-27	O. K.	Crazed	Crazed	Crazed
30/100	.50	1:3	6MR	15.40	3.36	110	1.02	17	Roof†	7- 2-27	Crazed	Crazed	Crazed	Crazed
		1:3	6MD	15.40	3.36	110	1.02	17	Dark closet*	7- 2-27	O. K.	Crazed	Crazed	Crazed
8/30	1.00	1:3	9ML	9.51	3.35	67	.96	16	Laboratory*	7- 5-27	O. K.	Crazed	Crazed	Crazed
		1:3	9MR	9.51	3.35	67	.96	16	Roof†	7- 5-27	Crazed	Crazed	Crazed	Crazed
		1:3	9MD	9.51	3.35	63	.96	16	Dark closet*	7- 5-27	O. K.	Slightly crazed	Slightly crazed	Slightly crazed
30/100	1.00	1:3	10ML	27.90	1.75	87	1.35	22½	Laboratory *	7- 5-27	O. K.	Slightly crazed	Crazed	Crazed
		1:3	10MR	27.90	1.75	88	1.35	22½	Roof†	7- 5-27	Crazed	Crazed	Crazed	Crazed
		1:3	10MD	27.90	1.75	88	1.35	22½	Dark closet*	7- 5-27	O. K.	O. K.	Slightly crazed	Crazed

* Removed to the roof 30 days after making.
† Started to craze from 10 to 15 days after making. See explanatory note under Table I.

made and stored for the first 30 days after making under three different atmospheric conditions: on the roof, in the laboratory and in a damp closet. At the end of 30 days all of the panels were placed on the roof. The marked difference in the deportment of the panels from the same mix made under the different conditions suggested that still other panels should be made which would be hardened under rather closely controlled conditions of temperature and humidity.

This latter set of panels includes Series N, P, R, and S.[2] The data regarding Series N and P are presented in Tables 4 and 5. It will be noted from these that the same eight mortars of Series M were used with the same water-cement ratio, but the panels immediately after making were placed for 72 hours in a chamber at either a temperature of 100 deg. F. and a relative humidity of 65 to 75 per cent (Series P), or at the same temperature for the same period but at a relative humidity of 40 to 50 per cent (Series N). In Series R there has been used a temperature for the first 72 hours of 85 deg. F., and a relative humidity of 65 to 75 per cent, and in Series S a temperature of 85 deg. F. and a relative humidity of 40 to 50 per cent. The panels in these series were maintained under these conditions for 72 hours and then one-half placed on the roof and the other half in the laboratory.

The above presentation of the extent of the research may leave an impression of its having been poorly planned. However, such is hardly the case. The original mortars were of such a gradation of aggregate that it was reasonably assured that some would craze and others not, under the conditions of interior storage. However, none of the panels developed the characteristic hexagonal crazing while specimens were left indoors. Hence, they were removed to the roof and other larger panels constructed. By the time the latter were made and under observation for some time in the laboratory the smaller panels on the roof were beginning to give evidence of failure. Hence the third series was inaugurated where larger panels were made in the different atmospheres. This behavior immediately indicated the need of curing under controlled humidity and temperature conditions. This is now being done.

Considering the data of Tables 1 and 2 and neglecting all except those on the mortar bases in the following discussion, it will be noted that the sand area-cement ratio varies from 2.9 to 29.1. The fineness modulus varies from 1.7 to 4.9. As long as the panels were in the laboratory no crazing resulted on those where the lower percentages of water had been used but after having placed these on the roof there was some slight evidence favoring the mixes with the high sand area-cement ratio. This tended to disappear with later aging but the mix with the highest ratio still shows the least crazing. Increasing the water by approximately 10 per cent did not produce any marked effect. Changing the mixes so that the sand area-cement ratio was about 12 to 13.7 slightly improved mixes

[2] Few data are presented regarding Series R and S, since the construction of the panels has just been completed.

TABLE IV.—SERIES N—FIRST 72 HOURS AT CONSTANT TEMPERATURE OF 100° F.

Sand		Mortar, Proportion by Weight	Mix Number	Surface Area, sq. in. per gr. cm.	Fineness Modulus	Flow, per cent	Water		Humidity, per cent	Date Made	Storage	Condition at End of First 72 Hours	Condition Oct. 14, 1927	Condition Nov. 14, 1927	Condition Jan. 14, 1928
Size	Proportion by Weight						*W/C*	Per Cent							
4/8 8/30 30/100	0.33⅓ .33⅓ .33⅓	1:3 1:3	1NL 1NR	13.45 13.45	3.35 3.35	90 90	0.90 .90	15.0 15.0	57 to 76 Av. = 63	8– 9–27 8– 9–27	Laboratory Roof	O. K. O. K.	O. K. Slightly crazed	O. K. Slightly crazed	O. K. Slightly crazed
4/8 8/30 30/100	.33⅓ 33⅓ .33⅓	1:3 1:3	1NL 1NR	13.45 13.45	3.35 3.35	90 90	.90 .90	15.0 15.0							
4/8 8/30 30/100	.60 .20 .20	1:3 1:3	2NL 2NR	9.22 9.22	4.00 4.00	93 93	.90 .90	15.0 15.0	57 to 76 Av. = 63	8– 9–27 8– 9–27	Laboratory Roof	O. K. O. K.	O. K. Slightly crazed	O. K. Slightly crazed	O. K. Slightly crazed
4/8 8/30 30/100	.60 .20 .20	1:3 1:3	2NL 2NR	9.22 9.22	4.00 4.00	93 93	.90 .90	15.0 15.0							
4/8 8/30 30/100	.20 .60 .20	1:3 1:3	3NL 3NR	11.87 11.87	3.52 3.52	85 85	.96 .96	16.0 16.0	30 to 55 Av. = 45	8–16–27 8–16–27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. Slightly crazed	O. K. Slightly crazed
4/8 8/30 30/100	.20 .20 .60	1:3 1:3	4NL 4NR	19.25 19.25	2.71 2.71	88 88	1.08 1.08	18.0 18.0	30 to 55 Av. = 45	8–16–27 8–16–27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.	O. K. O. K.
8/30 30/100	.50 .50	1:3 1:3	5NL 5NR	18.70 18.70	2.55 2.55	75 75	1.08 1.08	18.0 18.0	30 to 55 Av. = 45	8–16–27 8–16–27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.	O. K. Slightly crazed
4/8 30/100	.50 .50	1:3 1:3	6NL 6NR	15.40 15.40	3.36 3.36	93 93	.96 .96	16.0 16.0	35 to 58 Av. = 43	8–28–27 8–28–27	Laboratory Roof	O. K. O. K.	O. K. Slightly crazed	O. K. Slightly crazed	O. K. Crazed
8/30	1.00	1:3 1:3	9NL 9NR	9.51 9.51	3.35 3.35	80 80	1.02 1.02	17.0 17.0	35 to 58 Av. = 43	8–28–27 8–28–27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.	O. K. O. K.
30/100	1.00	1:3 1:3	10NL 10NR	27.90 27 90	1.75 1.75	87 87	1.35 1 35	22.5 22.5	35 to 58 Av. = 43	8–28–27 8–28 27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.	O. K. O. K.

See explanatory note under Table I.

TABLE V.—SERIES P—FIRST 72 HOURS AT CONSTANT TEMPERATURE OF 100° F.

Sand		Mortar, Proportion by Weight	Mix Number	Surface Area, sq. in. per gr. cm.	Fineness Modulus	Flow, per cent	Water		Humidity, per cent	Date Made	Storage	Condition at End of First 72 Hours	Condition Nov. 14, 1927	Condition Jan. 14, 1928
Size	Proportion by Weight						W/C	Per Cent						
4/8 8/30 30/100	0.33⅓ 0.33⅓ 0.33⅓	1:3 1:3	1PL 1PR	13.45 13.45	3.35 3.35	90 90	0.90 0 90	15.0 15.0	63 to 78 Av. = 70	0-14-27 9-14-27	Laboratory Roof	O. K. O. K.	O. K. Slightly crazed	O. K. Slightly crazed
4/8 8/30 30/100	0.60 0.20 0.20	1:3 1:3	2PL 2PR	9.22 9.22	4.00 4.00	93 93	0.90 0.90	15.0 15.0	63 to 78 Av. = 70	9-14-27 9-14-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.
4/8 8/30 30/100	0.20 0.60 0.20	1:3 1:3	3PL 3PR	11.87 11.87	3.52 3.52	85 85	0.96 0.96	16.0 16.0	63 to 78 Av. = 70	9-14-27 9-14-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. Slightly crazed
4/8 8/30 30/100	0.20 0.20 0.60	1:3 1:3	4PL 4PR	19.25 19.25	2.71 2.71	90 90	1.08 1.08	18.0 18.0	60 to 68 Av. = 65	9-19-27 9-19-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.
8/30 30/100	0.50 0.50	1:3 1:3	5PL 5PR	18.70 18.70	2.55 2.55	75 75	1.08 1.08	18.0 18.0	72 to 78 Av. = 75	10-10-27 10-10-27	Laboratory Roof	O. K. O. K.	O. K. O. K	O. K. O. K.
4/8 30/100	0.50 0.50	1:3 1:3	6PL 6PR	15.40 15.40	3.36 3.36	93 93	0.96 0.96	16.0 16.0	60 to 68 Av. = 65	9-19-27 9-19-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. Slightly crazed
8/30	1.00	1:3 1:3	9PL 9PR	9.51 9.51	3.35 3.35	76 76	1.02 1.02	17.0 17.0	72 to 78 Av. = 75	10-10-27 10-10-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.
30/100	1.00	1:3 1:3	10PL 10PR	27.90 27.90	1.75 1.75	80 80	1.35 1.35	22.5 22.5	60 to 68 Av. = 65	9-19-27 9-19-27	Laboratory Roof	O. K. O. K.	O. K. O. K.	O. K. O. K.

See explanatory note under Table I.

TABLE VI.—SERIES R—FIRST 72 HOURS AT CONSTANT TEMPERATURE OF 85° F.

Sand		Mortar, Proportion by Weight	Mix Number	Surface Area, sq. in. per gr. cm	Fineness Modulus	Flow, per cent	Water		Humidity, per cent	Date Made	Storage After 72 Hours	Condition After 72 Hours	Condition Jan. 14, 1928
Size	Proportion by Weight						W/C	Per Cent					
4/8 8/30 30/100	0.33⅓ 0.33⅓ 0.33⅓	1:3 1:3	1RL 1RR	13.45 13.45	3.35 3.35	87 89	0.90 0.90	15.0 15.0	63 to 84 Av. = 75	11- 2-27 11- 2-27	Laboratory Roof	O. K. O. K.	Crazed Crazed
4/8 8/30 30/100	0.60 0.20 0.20	1:3 1:3	2RL 2RR	9.22 9.22	4.00 4.00	88 91	0.90 0.90	15.0 15.0	63 to 84 Av. = 75	11- 2-27 11- 2-27	Laboratory Roof	O. K. O. K.	O. K. Crazed
4/8 8/30 30/100	0.20 0.60 0.20	1:3 1:3	3RL 3RR	11.87 11.87	3.52 3.52	86 86	0.96 0.96	16.0 16.0	63 to 84 Av. = 75	11- 2-27 11- 2-27	Laboratory Roof	O. K. O. K.	O. K. Slightly crazed
4/8 8/30 30/100	0.20 0.20 0.60	1:3 1:3	4RL 4RR	19.25 19.25	2.71 2.71	90 90	1.08 1.08	18.0 18.0	72 to 83 Av. = 77	11- 5-27 11- 5-27	Laboratory Roof	O. K. O. K.	O. K. O. K.
8/30 30/100	0.50 0.50	1:3 1:3	5RL 5RR	18.70 18.70	2.55 2.55	71 71	1.08 1.08	18.0 18.0	72 to 83 Av. = 77	11- 5-27 11- 5-27	Laboratory Roof	O. K. O. K.	O. K. O. K.
4/8 30/100	0.50 0.50	1:3 1:3	6RL 6RR	15.40 15.40	3.36 3.36	92 92	0.96 0.96	16.0 16.0	72 to 83 Av. = 77	11- 5-27 11- 5-27	Laboratory Roof	O. K. O. K.	O. K. O. K.
8/30	1.00	1:3 1:3	9RL 9RR	9.51 9.51	3.35 3.35	80 80	1.02 1.02	17.0 17.0	77 to 87 Av. = 83	11-15-27 11-15-27	Laboratory Roof	O. K. O. K.	O. K. Crazed
30/100	1.00	1:3 1:3	10RL 10RR	27.90 27.90	1.75 1.75	79 79	1.35 1.35	22.5 22.5	77 to 87 Av. = 83	11-15-27 11-15-27	Laboratory Roof	O. K. O. K.	O. K. O. K.

See explanatory note under Table I.

4, 6 and 9 on the larger panels only. This in effect increased the richness of the first two of these mixes and reduced the mixes of the last one. From the viewpoint of the gradation of size characteristics of the sand the data show that the more of the fine sand present the less there is of a readiness to craze. The fineness modulus seems to be more erratic as a means of decreasing crazing tendencies. The sand with the lowest modulus does show the least tendency but for any higher modulus there is no general relation. Thus the mix with the modulus of 3.35 is rather better than another with a modulus of 3.29. The small panels with the glass and rubber bases showed fine parallel cracks on drying generally normal to the direction of troweling. These would not have been called crazing cracks. On placing these panels on the roof crazing developed from these cracks very rapidly. The 24 x 24-in. panels on glass have been kept continuously in the laboratory and are without any evidence of crazing. They are not referred to in any of the tables.

The rapidity with which crazing resulted when the specimens were placed on the roof suggested the making and curing of some panels there. On doing this it was found that they crazed almost immediately and consequently Series N (Table 3) was started. The three atmospheric conditions under which the making and curing was done in this series gave immediately three different results but with age these differences have been considerably obliterated. It should be remembered, however, that the specimens of this series were kept in the atmosphere in which they were made for but 30 days after which all were placed on the roof. During the week in which they were made the temperature on the roof varied from 55 to 98 deg. F., but during by far the greater part of the time it ranged from 70 to 80 deg. F. The relative humidity varied from 25 to 90 per cent but generally it dropped daily to 30 per cent and rose during the night to 75 per cent. In the laboratory the relative humidity varied from 40 to 60 per cent and the temperature from 80 to 90 deg. F.

With this evidence of the marked effect of temperature and humidity during the early curing at hand, the several later series were started in which during the first 72 hours after making the panels were aged at one of three temperatures, 70, 85 and 100 deg. F., and at each of these temperatures two humidities are maintained, 40 to 50 per cent and above 70 per cent. After the first 72 hours half of the specimens were placed on the roof and the other half in the laboratory. Only parts of these series have been made. The others are in preparation at the present time. However, the effects of the control of these outstanding variables, temperature and humidity, are quite evident upon comparing the data of Series M and N. Whether this effect will persist over later ages time alone will indicate.

The data presented in Fig. 1 will be of value to those interested in volume changes. In obtaining this the several mixes were made in the form of slabs 1 x 4 x 24 in. in the laboratory and allowed to harden while measurements were made under an optical comparator. The data, how-

ever, do not serve very much in elucidating the crazing of the panels at the present time. It is possible that more indicative results would have been obtained if the panels had been made and measurements made during curing under controlled temperatures and humidities. This may be done later.

During the progress of the work two samples of commercial stuccoes were obtained which were reported to be showing no evidence of crazing in use. Panels were made of these on the mortar bases, in the air of the laboratory, in the damp closet and on the roof. After the first 30 days

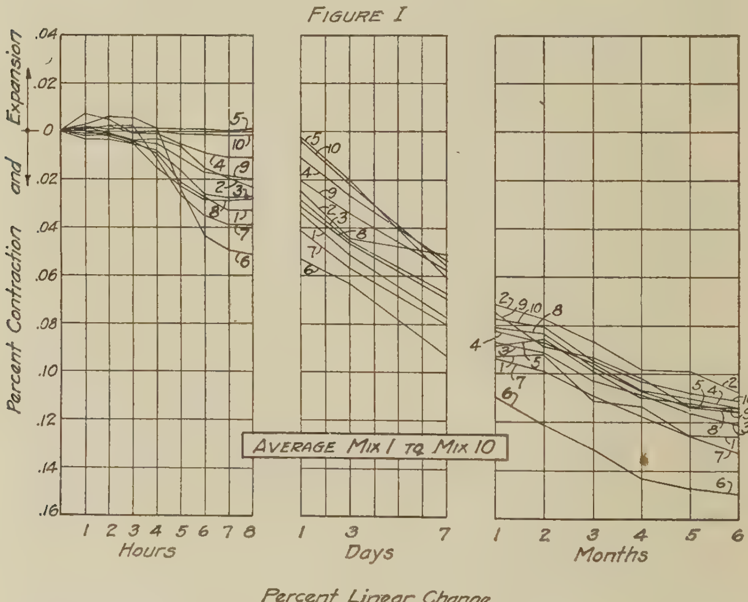

FIG. 1

all panels were put on the roof. These have been under observation for about three months. The panels of one of these to date show no crazing. It is beginning to be evident on the panels of the other indicated by the letter B in Table 7. The granulametric analyses of these commercial stuccoes as well as those of the mixes made at the Bureau are also given in Table 7. As the commercial stucco contained the cement an attempt has been made to obtain the gradation of the sands used by assuming that the material in them passing the 100 mesh is cement. By deducting this portion a granulametric analyses of the sands used is obtained. These values as indicated in Table 7 under the columns AA and BB show that they differ from any used at the Bureau. The linear shrinkage of bars from these was also determined under conditions similar to those obtain-

ing when the shrinkage of the bars of the ten other mixes was made. These specimens acted very similar to those of the laboratory mixes. Sometime during the first twenty hours of their hardening they showed an expansion of slightly more than 0.004 per cent. After the expansion a contraction resulted and at the end of 24 hours A showed a contraction of 0.051 per cent and B 0.038 per cent. At the end of the first week the contraction of the former was 0.111 per cent and of the second 0.139 per cent. These values are higher than those obtained on any of the bars made from the mixes prepared at the laboratory.

It is confessed that so far, after a year and a half of work, nothing very tangible for solving the problem of crazing has been obtained but on the other hand apparently an estimate of the magnitude of the effect of some of the several variables, particularly humidity and temperature, in causing this annoyance is being obtained. This is of much importance and

TABLE VII.—GRANULARMETRIC ANALYSIS OF THE SANDS USED

Per Cent Retained on Mesh No.	Mix Number													
	A	AA	B	BB	1	2	3	4	5	6	7	8	9	10
4	0.0	0.0	0.0	0 0	0.4	1.3	0.4	0.2	0.0	0.6	0.6	0.0	0.0	1.0
8	0.0	0.0	2.0	3.0	31.6	56.2	19.0	19.6	0.4	47.6	47.8	0.0	0.2	94.0
16	8.1	13.3	11.8	17.5	12.2	9.6	19.8	7.3	16.5	1.8	17.6	0.0	32.0	4.8
28	23.3	38.4	26.6	40.4	20.9	12.0	37.2	12.6	30.8	0.6	31.0	0.8	62.0	100.0
48	18.2	30.0	21.8	32.4	26.9	15.4	17.4	46.4	39.6	37.8	3.0	75.4	5.4	.....
100	11.0	18.1	4.6	6.7	7.8	5.4	5.8	13.6	12.6	11.4	0.1	23.6	100.0	.....
200	7.0	100.0	6.4	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	.....	.....
Passing 200	32.3	.....	26.6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

A and B = Prepared commercial stuccoes.
AA = Calculated analysis of sand of A on the basis that all of A passing 100 mesh is cement.
BB = Calculated analysis of sand of B on the basis that all of B passing 100 mesh is cement.

will assist materially in leading the present and future work more directly toward solving the crazing problem. For the immediate future the work will be conducted along the line adopted in the latter series of panels. These data when obtained should indicate which of the stuccoes are distinctly better than others under the most severe conditions such as high temperature and low humidity and under the most favorable conditions as lower temperatures and higher humidities. Then it would be desirable to study other mortars of different gradations of sand. In the study of this latter variable the field is so broad that for all preliminary work it was thought desirable to use a limited number of carefully graded sands. It might also be advisable to use different cements. This was not done in the present case largely because no test method was available or known that would indicate the difference in cements of their tendency to assist in producing crazing but before any enlargement of the work to include sand gradations or different types of cement is undertaken suggestions are especially invited regarding both of these variables.

Crazing in Concrete and the Growth of Hair Cracks Into Structural Cracks.

By Alfred H. White,[1] Vilhelm A. Aagaard, and Axel O. L. Christensen.

The fine cracks which occur on the surface of concrete and especially of mortar rich in portland cement are frequently called hair cracks and the phenomenon is often alluded to as crazing. A crack may result from a blow or other mechanical cause, but the most frequent cause is the unequal expansion and contraction of the surface and the main body of the material. This unbalanced condition may be caused by heat, and temperature changes are undoubtedly a contributing cause, but the chief cause of both the initial formation and the growth of these cracks lies in variation in the moisture content of the hydrated cement.

Colloidal Properties of Hydrated Portland Cement.—Portland cement, after hydration, is a mixture of crystalline and colloidal materials, and the colloidal material expands when it is wet and contracts when it is dry as is the rule with colloidal gels. This property of behaving as a colloid is not lost with age as is shown in Fig. 1, which records the changes in length of four bars of neat cement approximately 4 in. long 1 in. in cross section when alternately wet and dried during a period of more than 18 years. The two upper curves represent the behavior of two commercial cements, passing standard specifications, which were each mixed with water to normal consistency and made into an expansion bar. These two bars 131 E and 131 G were kept continuously in water for nearly three years as shown in Fig. 1. They increased in length for the first year and then remained substantially constant in length for the two following years in water. When they were allowed to dry in air, each one shrank about 0.12 per cent, and when they were wetted again they expanded. This change in length has not only persisted with each complete reversal of moisture content, but the swings have increased in amplitude with the years. In the last change from the dry to the wet state shown on Fig. 1 between the seventeenth and nineteenth years the expansion was 0.27 per cent for 131 E and 0.31 per cent for 131 G.

[1] Professor of Chemical Engineering, University of Michigan.

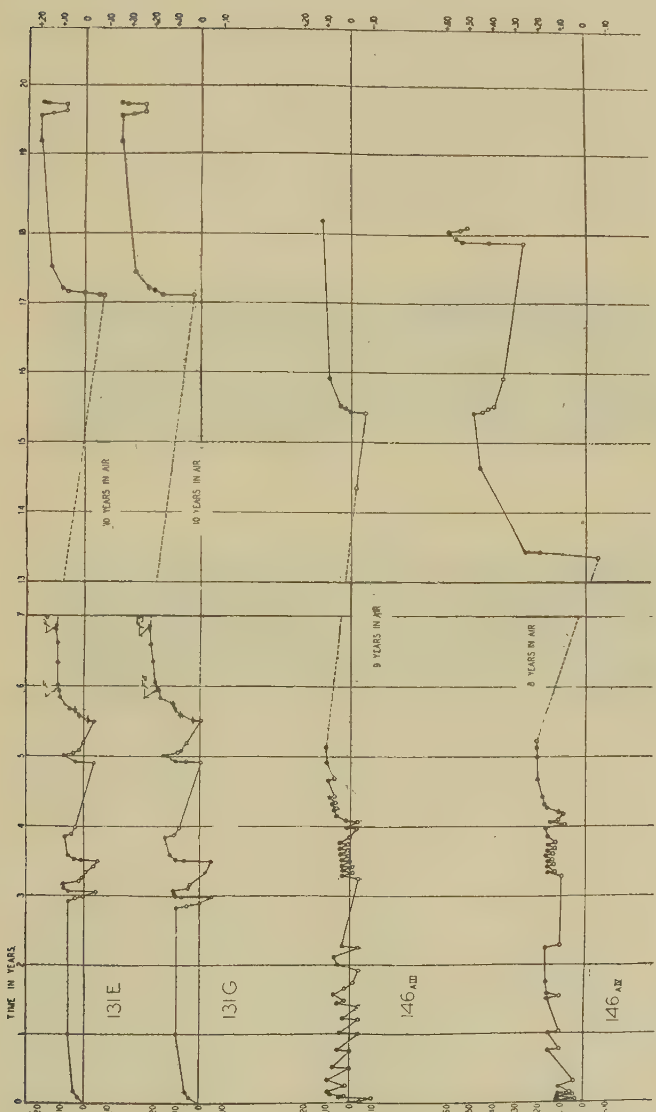

FIG. 1.—PERCENTAGE CHANGE IN LENGTH OF NEAT CEMENT BARS WHEN ALTERNATED BETWEEN THE WET AND DRY CONDITION.

The ordinates express percentage in hundredths of a per cent. The full circles show immersion in water, the hollow circles exposure to dry air, and the full circles with a horizontal line show exposure to damp air.

The effect of damp air is also shown in the history of these two bars, between the fifth and sixth years. Bar 131 E when dry was exposed at room temperature to air almost saturated with water vapor, and expanded 0.08 per cent in length in three months. Bar 131 G under similar circumstances expanded 0.12 per cent in four months.

The two lower bars of Fig. 1, 146 A III and A IV were made in a similar way from another commercial cement of good quality. Their history differs from that of the other two in that they were alternated more frequently between the wet and the dry states. There is a difference in the behavior of A III as compared with A IV which is the result of the plan

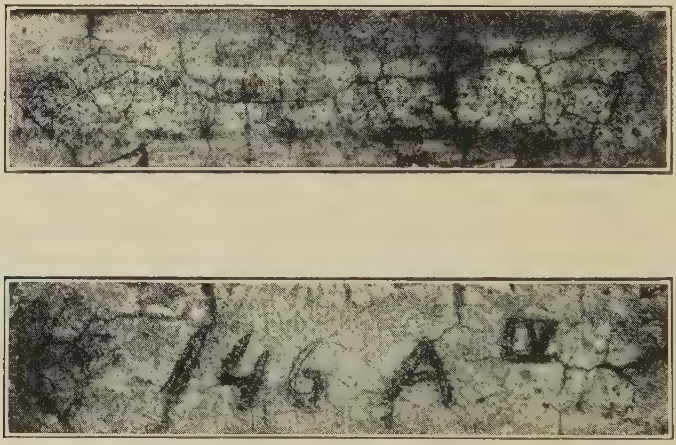

FIG. 2.—PHOTOGRAPH OF BAR OF NEAT CEMENT AFTER HAVING BEEN EXPOSED TO ALTERNATE WET AND DRY CONDITIONS AT ROOM TEMPERATURE FOR 18 YEARS. THE VOLUME CHANGES OF THIS BAR ARE SHOWN GRAPHICALLY IN FIG. 1.

laid out when the bars were made. Both bars were to be alternated between air and water but A III was placed in air after the first day and was given relatively long periods in air, while A IV was placed in water after the first day and given long periods in water. This plan resulted in A IV growing steadily while A III fluctuated, but maintained its average length about what it was initially, during the first four years. After that period, A III was changed to longer periods in water and it grew somewhat under this treatment. Its length after 18 years, the last two of which were in water was +0.12 per cent while A IV at a corresponding period was +0.60 per cent. Each of these four bars is crazed, but A IV is now so badly and deeply cracked that it seems almost ready to go to pieces. Its appearance is illustrated in Fig. 2, which shows the deep

cracks in the top and in one side. It will be noticed that the corners of the bar have spalled in three places.

The explanation of the increasing amplitude of the variations in length as the bar becomes older has been discussed elsewhere.[1] It is sufficient to say here that it is due to the incomplete hydration of the particles of portland cement even after exposure to water for several years.

In a rich concrete which is well made and kept wet there is slow expansion which terminates only when the products of hydration have become packed so closely that no more water can force its way to the unchanged clinker. This is shown diagrammatically in Fig. 3. In A, four sand grains are represented on a magnified scale, bonded together by a

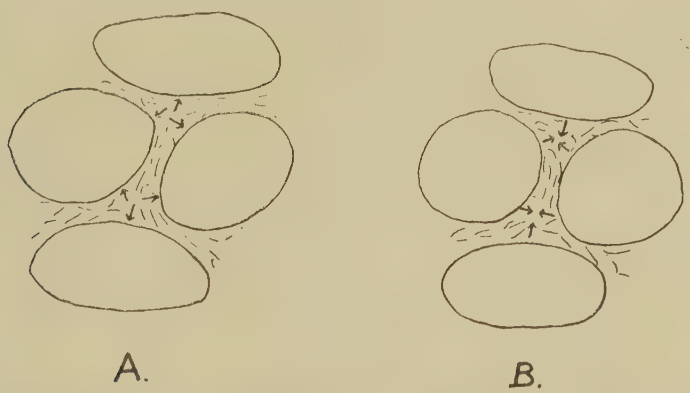

FIG. 3.—DIAGRAMMATIC REPRESENTATION OF STRESSES IN CEMENT MORTAR WHEN WET AND DRY.

A represents four sand grains with the colloid between them wet and swollen. The arrows show that the grains are being pushed apart, and that the colloidal cement is in compression: B represents the same situation with the colloid dry and contracted. The grains are being pulled together and the colloidal cement is in tension.

colloidal cement which is swollen and which tends to push the sand grains apart, the forces being represented by arrows. A bar of cement mortar or concrete when wet is therefore elongated and if there is any restraint, each film of cement is in compression. In B of Fig. 3 the same grains of sand are represented after the cement has become dry. The colloidal material has shrunk and is pulling the sand particles together. A bar under these circumstances contracts and if there is any restraint, each film of cement is in tension.

When water starts to evaporate from the surface of a wet bar, the surface tries to contract, but is restrained by the swollen material below it. The bottom layers are therefore placed in compression and the top layers in tension. Since cement is much stronger in compression than in tension,

[1] Colloid Symposium Monograph, Vol. VI, 1927.

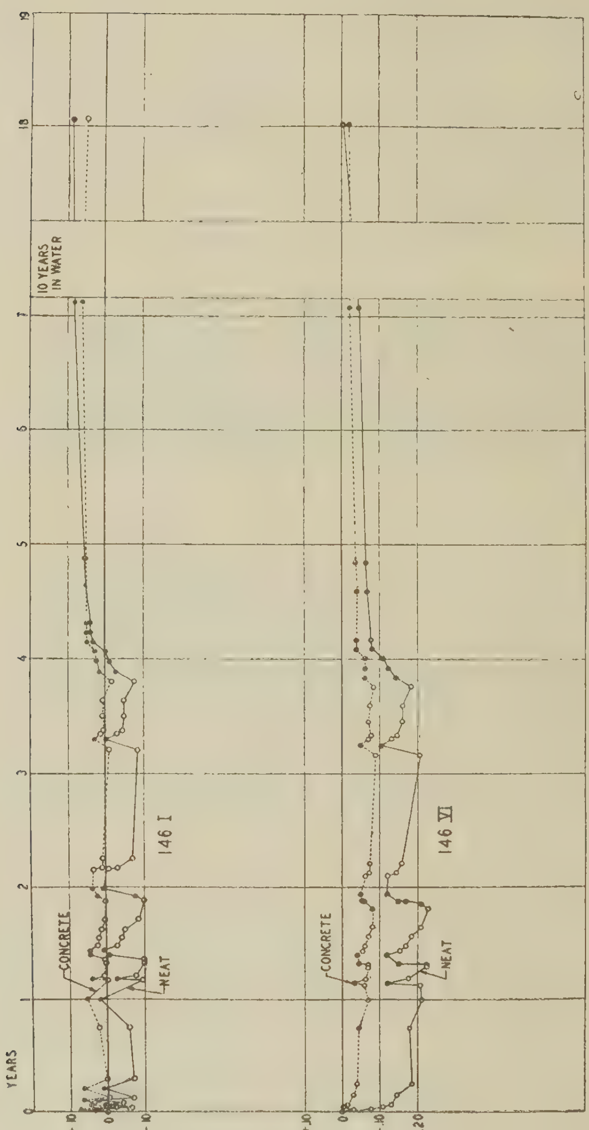

FIG. 4.—PERCENTAGE CHANGE IN LENGTH OF COMPOUND BARS WHEN ALTERNATELY WET AND DRY.

The solid line shows the variation in the neat cement portion and the dotted line the variation in the 1:3 portion of the compound bar. The full circles indicate immersion in water, and the hollow circles exposure to air.

there will be a tendency for cracks to form on the surface which is in tension. If cracks do form on the surface they will penetrate until a layer is reached where the tension becomes too small to cause rupture. The magnitude of the shrinkage in the upper layers will depend on a good many factors. It will increase with the quantity of colloidal cement present and with the rate of evaporation of water from the surface. It will be smaller with lean mixes and with slow drying, which will permit the moisture to be transferred from the bottom to the top layer almost as fast as it evaporates from the surface.

The expansion and contraction of hydrated cement with moisture changes is apparently universal. It does not follow, however, that there will be permanent expansion over the initial length. The four bars of Fig. 1 all show permanent expansion because they have been kept wet for long periods. The bars of Fig. 4 show similar volume changes, but their length has been in general less than their initial length because they were allowed to become dry soon after they were made and were kept dry for relatively long periods and wet for shorter periods, during the first few years of their life. The amplitude of the volume changes of the neat cement portion of the compound bars of Fig. 4 is substantially the same as in the free bars of Fig. 1.

Volume Changes in Two-Course Structures.—Pavements and sidewalks are frequently made in two courses, a thick and rather lean base with a thin and rather rich topping. The richer top coat crazes when exposed to the weather and the extent of the cracks is influenced to a material extent by the relative inertness of the lean base. Fig. 4 represents a laboratory experiment intended to show the relative volume changes of such a compound structure. Bars 146 I and 146 VI are compound bars made by superimposing a bar of neat cement on one made of a 1: 3 mix. These bars were as usual 4 in. long by 1 in. square and were superimposed on one another when they were formed, so that they bonded together firmly.

Bar 146 I was alternated between water and air on a weekly schedule for the first six weeks and was then given longer alternations. Bar 146 VI was stored in air for 14 months and was then started on a schedule alternating between air and water. It will be noted that the shrinkage of the neat cement was much greater than that of the 1: 2 mortar so that there must have been a greater tendency for the neat cement to crack in tension as it became dried. With each immersion in water the greater tendency of the neat cement to elongate lessened this difference until at the end of four years both parts of the bars were of nearly equal length. They were then allowed to lie in water for 14 years and at the end of this period the neat cement was in both cases longer than the 1: 3 mortar portion of the bar. The relative stresses in the two bars have thus been reversed. Each bar of neat cement shows numerous hair cracks but the bars are still bonded together.

Experimental Study of Formation of Hair Cracks.—Hair cracks are primarily due to unequal shrinkage. In the usual case of pavements and

stuccos they are caused by too rapid drying of the upper surface. If the drying continues after the hair cracks are formed, and the whole mass comes to uniform moisture content, the strains are relieved in part and if the mass is unrestrained the shrinkage cracks will close. If the shrunken mass is exposed to water working from the upper surface, the upper layers will expand and the hair cracks will close tightly under compression. They

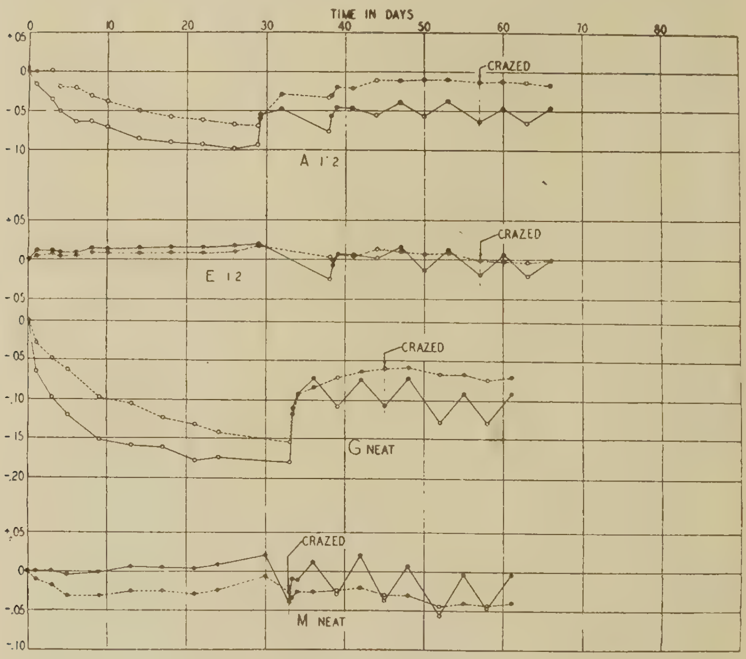

FIG. 5.—INITIATION OF CRAZING IN BARS OF NEAT CEMENT AND 1 : 2 MORTAR AS THE RESULT OF ALTERNATIONS BETWEEN THE DRY AND WET CONDITION.

The vertical lines show time in days, the horizontal lines percentage change in length, in hundredths of a per cent, of bars of Series 192. The full lines of the curves show the variations in length of the upper portion of each bar. It was through the upper surface that moisture entered and left the bar. The dotted lines of the curves show the variations in length of the lower portion of each bar which changed its moisture content only slowly.

will remain closed until the mass dries again from the top surface, when the cracks will open again and will penetrate more deeply into the body of the structure. Repeated alternations of moisture content may therefore cause the cracks to penetrate very deeply, as is shown in the photograph of bar 146 AIV of Fig. 2. All of the bars of neat cement which have been alternated between air and water many times in our laboratory show crazing. The older records do not show when the first hair cracks were

observed and a new series of bars was made up to test the effect of relatively rapid alternations of exposure to air and water.

The changes in moisture content of a concrete pavement or sidewalk come mainly through the upper surface and in order to obtain a measure of the variations in volume which might result at short distances below the surface the bars of series 192 were made. Six bars, A to F inclusive, were made from a 1:2 mixture by weight. The sand was of good screen analysis with nothing that would not pass a 10-mesh screen, and 11.9 per cent of water was used. The bars were made 4 in. long and 1 in. square with glass plates 1 in. long embedded in each end. They were kept in the molds in the damp box for 24 hours and as soon as the molds were stripped all of the faces, except the top, were painted with a thick coat of paraffine, so that absorption or evaporation of water would take place almost entirely through the top surface. Measurements of the length of the bars was made at two points, about ¼ in. from the top and ¼ in. from the bottom, and the initial measurement of each bar was plotted as zero.

The changes in length of these bars as changed from water to air and back to water is shown in Fig. 5. The full lines show the changes in the upper part of the bar which changed its moisture content rapidly, and the dotted lines show the variations in the bottom of the bar whose moisture content changed more slowly, because of its paraffine coating. The moisture from the lower part of the bar had to pass to the upper surface by diffusion and be evaporated there. Bar 192 A, of 1:2 mix was kept in the air of the laboratory for 28 days. The upper part of the bar showed an evident shrinkage after the first 24 hours in air, but the bottom of the bar retained its initial length even after three days in air showing that it had not yet started to dry out. After the third day, the evaporation of water from the surface could apparently proceed only as fast as water came to it by diffusion from the lower portions of the bar, for the top and bottom both contracted at substantially the same rate until, after 28 days, the bar was placed in water. After three hours in water the upper portion had elongated 0.031 per cent while the bottom had only elongated 0.002 per cent. After three days in water the upper portion elongated an additional 0.016 per cent while the bottom elongated an additional 0.039 per cent, making a total expansion of 0.047 per cent for the top and 0.041 per cent for the bottom. The bar was then placed alternately in air and water for periods of three days with results shown in Fig. 5. The length of the lower part of the bar protected by paraffine remained almost constant whether immersed in water or in air since the interval of three days in air was too short to permit the evaporation from the upper surface to affect the moisture content of the lower portion. The micrometric measurements of length at points ¼ in. below the upper surface, however, fluctuated about 0.03 per cent with each three-day change of state and actual surface of the bar must have been subjected to greater and more rapid fluctuations. As the bar dried after its fourth immersion in water and after a total life of 56 days the first faint hair cracks became visible, and these became more evident with each successive alternation.

Bar 192 E was made as a duplicate of 192 A but was kept in water for the first month after it had been coated with paraffine. It was then alternated between water and air on a three-day schedule and after the fourth period in air it was found to have hair cracks.

Bar 192 G was made from neat cement, coated all over, except its upper surface, with paraffine and placed on the same schedule as Bar A. It shrank nearly twice as much as A during the first month in air and the

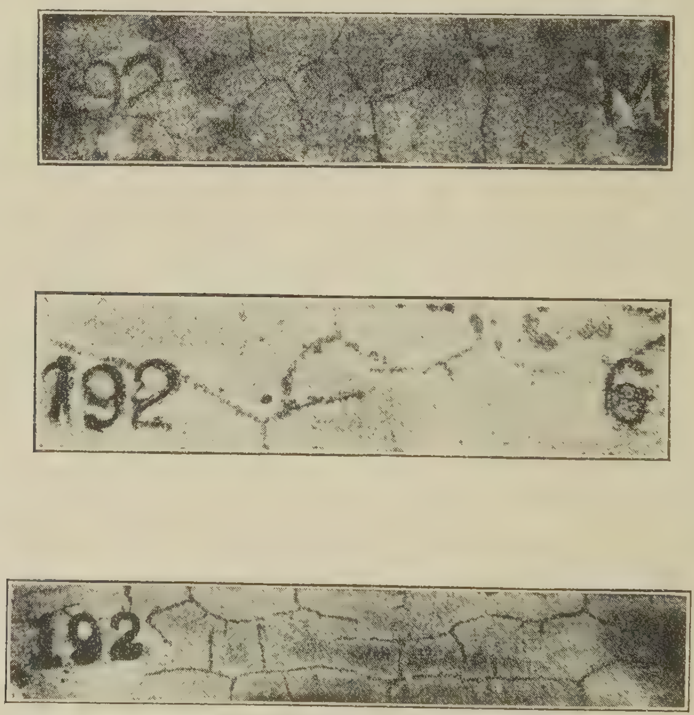

FIG. 6.—CRAZING IN NEAT CEMENT BARS DUE TO ALTERNATION BETWEEN WET AND DRY CONDITIONS AT ROOM TEMPERATURE.

fluctuations in length were 50 per cent greater as it changed from the wet to the dry state. The lower part of the bar protected by paraffine varied little in length during the three-day alternations while the upper changed 0.03 to 0.05 per cent. Crazing was visible on this bar as it dried for the second time, after a total life of 45 days.

A similar history could be written about the bar of neat cement 192 M which received the same treatment as 192 E. It, however, showed crazing the first time it dried out. The photograph of bar 192 M shown in Fig. 6

was taken after it had dried out for the third time when its total life was 45 days. The bar was dipped into water just before the picture was taken so as to show the cracks more clearly. These experiments showed that an unrestrained bar of neat cement or one of 1 : 2 mixture would craze after a few alternations between the wet and dry states at room temperature, if evaporation and absorption of water could take place only from the top surface.

Experiments having shown that neat cement bars or bars of 1 : 2 mixture protected from moisture from all sides but the top, all crazed when alternately dried in air and immersed in water, it was desirable to find how bars behaved when free to absorb water on all surfaces. It seemed probable that bars of small cross section would have an advantage in that the water would penetrate to the center more rapidly and so would equalize the stresses more rapidly. Accordingly bars of neat cement, 1 in. square and 1½ in. square were fabricated and allowed to remain in the molds in damp air for 24 hours. They were then placed on a schedule of air 24 hours and water 24 hours. Two of the six bars showed crazing after the seventh alternation and all of the bars showed crazing after the ninth alternation. The photographs of 192-6 and 8 in Fig. 6 were taken after the tenth alternation. The bars were dipped in water just before photographing in order to show the cracks more clearly and this accounts for the somewhat hazy outline of the edges of the crack. Bar 6 is one of the bars only 1 in. square and bar 8 is one of the larger bars 1½ in. square.

The Growth of Hair Cracks into Structural Cracks.—The discussion up to this point has dealt with the formation of minute cracks on the surface. The present discussion deals with the spread of these cracks into the body of the material. When concrete dries rapidly, the greatest shrinkage stress is manifested at the surface and is, theoretically, uniform over every part of the surface. As soon as a crack is formed, the stresses are no longer uniform but are greater at the bottom of the crack. A force which will produce an initial crack will therefore be great enough to cause it to spread in a homogeneous material. This is well illustrated by the manner in which a crack spreads in a pane of glass.

When concrete dries through rather rapid evaporation of water from its surface the stresses are not only greatest at that surface, but they reach their maximum at some particular time. When the concrete is thoroughly wet, the surface as well as the rest of the mass is in compression. As it dries, the surface gradually passes into tension while the base usually remains wet and in compression. Ultimately the base also passes into tension if the concrete becomes completely dry and the tension of the surface layer becomes relieved somewhat. There is a particular moment during the drying process when the stresses are greatest. A crack may thus start at the surface and make relatively little progress during a single drying. The crack will become deeper with each alternation between the wet and the dry state, and may ultimately develop into a complete structural crack.

Crazing may take place from any surface from which there is evaporation. In practice there is usually only one surface exposed to the air, but in laboratory tests on bars exposed to evaporation from all surfaces, crazing takes place on all surfaces even with bars as small as 1 in. square.

The development of hair cracks is naturally greatest in rich mixtures because the shrinkage is due to the colloidal properties of the hydrated cement. The growth of the hair cracks is also most rapid in rich mixtures, not only because of the greater amount of colloidal material, but because of the greater continuity of the colloid. A hair crack in a lean mixture may be stopped when it meets a particle of aggregate. A surface which has been trowelled is richer in cement and is therefore more likely to crack. It is also smoother and more homogeneous so that the crack is more easily visible and spreads more rapidly. The failure to observe crazing in lean mixtures is frequently due to the irregular outline and lack of continuity of the cracks.

Hair Cracks Do Not Repair Themselves.—Hydrated cement is plastic and colloidal, and there is some evidence that minute cracks forming in the interior of a concrete mass may repair themselves. This does not happen, however, with cracks originating at the surface because the carbon dioxide of the air reacts with the hydrated lime present in all hydrated cement and forms crystalline calcium carbonate over the fresh surface.

Crazing as an Initial Cause of Disintegration of Concrete Exposed to the Weather.—Hair cracks are formed initially because of shrinkage of the surface while the main portion of the concrete is wet and swollen. Cracks thus formed may become closed when the surface becomes wet and swollen but always reopen when the surface becomes dry. They increase in size with each alternation from a wet to the dry state. Crystals of calcium carbonate forming in the cracks, and dust which works into the open cracks, prevent complete closure even when the concrete is wet and swollen. Under favorable circumstances, ice will form in the cracks and open them still further. The disintegration proceeds at an accelerated pace as the cracks enlarge. Temperature changes may accelerate the growth of cracks but are, in general, of secondary importance. A change in volume of 0.05 per cent in a rich mortar may be caused by a rain continuing only for a few hours. This is as much as would be caused by a temperature change of 100 deg. F.

Two-course sidewalks frequently show failure between the two courses. The compound bars of Fig. 4 indicate the difference in shrinkage between two layers of unequal richness, giving rise to a stress which tends to split the two layers apart. In sidewalks which failed in this manner, it is probable that hair cracks in the top coat progressed until they became structural cracks extending through the top coat, and the top split from the bottom because of the differential expansion and contraction of the two layers. Poor bonding, temperature effects, and freezing may assist in bringing about such a result, but it is not necessary to assume their aid.

Prevention of Crazing.—The experiments which have been discussed

show that crazing is caused by shrinkage of a cement surface which is drying faster than the main body of the concrete. It is not possible to change the colloidal properties of the hydrated cement but it is possible to diminish shrinkage by using a lean mixture. A lean mixture has two advantages: There is less colloidal cement to cause shrinkage, and the absorption of water through a lean mixture is much more rapid than through a rich mixture. The whole mass of a lean mixture expands and contracts more as a unit. On the other hand, a lean mixture may be so porous that it will disintegrate through freezing of the water contained in the pores.

If a rich mixture must be used, an attempt should be made to protect it so that it will change its moisture content slowly and as infrequently as possible. Protective coatings are helpful. Integral waterproofings which are water-repellant are also helpful. As was shown in a paper in the 1926 *Proceedings* of the American Concrete Institute[1] very small amounts of insoluble soaps are effective in reducing water absorption, and if the concrete containing these soaps is given an opportunity to gain strength before it is dried out an entirely satisfactory strength may be developed.

Conclusions.—Crazing is due mainly to the rather rapid evaporation of water from the surface of a rich cement mortar or concrete. The surface becomes dry and tries to contract but is restrained by the main body of the material which is still wet and swollen. The surface is thus put into tension and hair cracks form. The colloid of cement requires many months in water for its full development and the magnitude of the stresses, caused by alternations of wet and dry conditions, increases with the amount of colloid developed. The fact that hair cracks cannot be detected after the first alternation between the wet and dry state does not indicate that they may not be formed later. The same forces which cause hair cracks to form are also adequate to make them larger and deeper so that hair cracks may ultimately become structural cracks.

Crazing may be prevented by using lean mixtures, and by keeping the moisture content of the concrete constant. It is immaterial whether the concrete is wet or dry so long as the moisture is constant. Protective coatings and integral waterproofings of the water-repellant type are helpful. If integral waterproofings are used, care must be taken to keep the concrete damp until adequate strength is obtained, for after such waterproofed material has once become dry, it will absorb water only slowly.

[1] White and Bateman—Soaps as Integral Waterproofings for Concrete. *Proceedings* American Concrete Institute, 22, 535 (1926).

Discussion—Crazing of Concrete.

Mr. Mills.

R. E. Mills.*—It has been realized for some time that volume changes of cements and concretes are of great importance. However, reliable data concerning this phenomenon are few. Since the permanence of concrete is largely dependent upon the preservation of an integral surface, a more complete study of the effect and extent of the volume change is essential. Considerable data on crazing with especial regard to surface fissures appear in a paper in the 1926 *Proceedings*.[1]

Two series of neat cement beams (2 in. x 2 in. x 24 in.) have been under observation and exposed to the air of the testing materials laboratory, Purdue University, for about 3½ yrs. Although the total life of these specimens is only about 1/5 that of Professor White's specimens (Fig. 1), many of the characteristics which he discusses, with respect to volume change and crazing were found in the Purdue tests.

All of these neat specimens developed considerable crazing during their initial drying out period of about 100 days following a period of moist curing. In the case of the first series of beams which have remained in the air of the laboratory, this crazing has progressed slightly to date. However, the second series of beams which has remained in water after 220 days of drying in the air of the laboratory are firm and show no further indication of disintegration.

Volume Change Neat Cement—The first series of tests of the two referred to above records the change in length of neat cement beams made from six different brands of cement. Each brand filled the requirements of standard specifications. The beams have been exposed to the air of the laboratory for a period of 1333 days to date, subsequent to 10 days initial curing in water. The initial readings were taken 20 hr. after moulding. The individual behavior of the several brands of cement is confined to the first period of about 100 days, after which the movements of the several brands are approximately alike. The maximum initial contraction varied from about 0.12 per cent to 0.23 per cent, giving a range of approximately 0.11 per cent for the various brands of cement. The beams seem to follow very closely the seasonal changes of the year, showing contraction in the warm dry air of the laboratory during the winter months, and expansion in the moist air and atmospheric temperatures of the summer months. The average total amount of volume change during the year is about 0.05 per cent.

*Research Assistant, Engineering Experiment Station, Purdue University.
[1]Extensibility of Concrete by W. K. Hatt. *Proceedings*, American Concrete Institute, Vol. 22, 1926.

A close inspection of the individual plotted observations indicates a small amount of permanent expansion, or growth to be taking place. All of the curves of expansion plotted from the data show approximately 0.02 per cent of expansion during the last 1,000 days.

The second series of tests of neat cements represents a sampling of the market six months later than the first series of tests. It appears that the several brands of cement maintain their relative position.

The beams in this second series were allowed to cure for a period of 50 days in water, after which they were subjected to the dry winter air of the laboratory for 220 days. By the end of this drying period their contraction values varied from about 0.06 per cent to 0.15 per cent, giving a range of approximately 0.09 per cent for the various brands of cement. The specimens were then submerged in water and their expansive movements observed. To date they have been in water for a period of 889 days and still show a consistent expansive movement. The average total expansion to date is about 0.20 per cent, and the range between various brands of cement is approximately 0.11 per cent.

All of the values quoted for the Purdue tests are for volume changes following changes in moisture content only. Corrections have been applied for any variations in temperature or variations in the measuring equipment. No indication is made for the values reported by Professor White, whether they are the result of moisture changes alone or represent the effect of the combined variables.

All of the data and observations reported by Professor White, as well as the Purdue tests so far discussed, deal either with neat cements or rich cement mortars.

Similar tests of concrete have been under way at the Testing Materials Laboratory, Purdue University, for approximately 2½ years. Observations on the crazing of one series have appeared in the *Proceedings.*[1] Observation for 30 years of concrete walls show no evidence that surface crazes will necessarily develop into fissures.

Volume Changes of Concrete.—In another series of tests on concrete, observations of volume change were recorded upon concrete beams (4 in. x 7 in. x 48 in.) made from three different coarse aggregates; gravel, limestone and slag. The concrete was mixed in proportions of 1:2:3, 1:2½:4, 1:2½:5 and 1:3:6. Two beams, both plain sections, were made for each condition of test. Additional blank beams were made from an average mix of each coarse aggregate in which thermometer wells were provided for obtaining temperatures of the concrete.

The beams were removed from the forms 3 days after pouring. The gage plugs were then set, and the initial readings taken. The specimens were then placed between layers of wet burlap on the floor of the cement laboratory. After 14 days of such curing they were set upright in the air of the laboratory and allowed to dry for a period of 182 days. At

[1]Extensibility of Concrete by W. K. Hatt, *Proceedings*, Concrete Institute, Vol. 22, 1926.

the end of this time their contraction movement had practically ceased. The beams were then immersed in water and saturated to a constant weight. The time required for this period was 130 days. This saturation in turn was followed by a second saturation period of 106 days, thus making two complete cycles of drying out to constant weight and saturation to constant weight.

Observations of change in length and change in weight of the beams were made at intervals, depending upon the rate of change of the contraction or expansion movements. The measuring equipment consisted of a 20-in. Berry strain gage equipped with a 1/10,000 Ames dial. Corrections to these measurements on account of temperature variations were made on the basis of a temperature coefficient of 0.0000050 per deg. F.

The various concretes show an average value of contraction for the first drying period of 0.0389 per cent; this was followed by 0.0259 per cent expansion for the first saturation period. The second drying period followed by a second saturation period yielded values of 0.0223 per cent and 0.0213 per cent respectively. This would indicate that for long period was then followed by a second drying period of 255 days, which periods of drying followed by long periods of saturation in water the total expansion is less than the total contraction, leaving a residual shrinkage. While there is a residual shrinkage after a cycle of drying and saturation, no change in weight is indicated during the same cycle. Similar characteristics of concrete tested at the University of California are reported by Prof. Raymond E. Davis. This residual shrinkage of concrete would indicate a direct reversal of the results found in the case of neat cement. All of the neat cement tests show a consistent growing or expansive movement of the specimens when subjected to conditions of drying and saturation similar to that of the concrete.

It is further evident from the various observations that the richness of mix has some effect on the volume changes of concretes. The richer the mix the greater the volume change. However, the changes in weight of the concretes is greater for the leaner mixes, although the differences between the various mixes is less pronounced than it was for the change in length. For example: A concrete of a 1:2:3 mix showed a contraction of 0.0438 per cent at the end of 182 days of drying in the air of the laboratory. A 1: 3: 6 concrete showed a contraction of 0.0345 per cent for the same drying period. Corresponding values for loss in weight were, (1: 2: 3 mix) 3.14 per cent and (1: 3: 6 mix) 3.63 per cent. Similar differences between mixes were found for the saturation periods, the richer concretes expanding most and the leaner concretes showing the greatest gain in weight.

It is evident from this series of tests that concrete does not exhibit the same characteristics with respect to expansion and contraction movements as does neat cement, and perhaps many of the other attributes of concrete are also at variance. A complete study of the cement is no doubt

an important step in the solution of many of the problems in concrete. However, any attempt to express the action of concrete in terms of neat cement, or rich cement mortars alone might prove erroneous or misleading.

No doubt other factors with respect to the aggregate have an important influence upon the volume changes of concrete. It has been noticed in other tests that the character of the aggregate as well as the graduation of the aggregate seems to exert an influence upon the resulting volumetric changes. A more complete study of these several variables should prove a fruitful field for future research.

F. N. MENEFEE (*By letter*).—The writer has long been interested in Professor White's studies of concrete, and has read this new paper with considerable interest, adding the following by way of corroboration and substantiation, but from a different point of view: **Mr. Menefee.**

All water above the necessary amount to properly hydrate the cement, is theoretically excess. Any water above that amount is free water and under some conditions is apt to dry out. If it drys out when the concrete has set, shrinkage and microscopic porosity are the result.

Shrinkage is the cause of the hair crack or crazing, and crazing is a good place for moisture from the elements to enter, thus producing its attendant corrosive effects, with varying degrees of undesirability.

One way of getting a qualitative idea of the process which takes place in the drying out of the free water in concrete is to compare its surface with the surface of the bottom of a pond from which the water has evaporated, leaving the mud exposed. As the water recedes from the surface, capillary attraction of the water pulls in the direction of the receding moisture. It pulls down vertically, causing the surface layer of clay to become thinner, and it pulls horizontally, causing a shortening in the horizontal plane as well. Having little tensile resistance, the clay cracks up into irregular shaped areas a few inches across. Each of these little areas continue to shrink in all directions toward the center of moisture. Naturally it must shear away from the layer immediately underlying it, which as yet has not lost its free water. The upper or exposed surface is pulled downward and horizontally toward the center of moisture. It responds to the pull by shortening more than the bottom which is still wet and relatively unshrunk, and a saucer-like piece of clay results. The same process, although to a lesser degree to the naked eye, takes place in the drying of concrete and is the cause of crazing. It is the reason why sidewalks made by using a dry lean base and a rich wet top eventually separate along the plane between the two courses by shear due to shrinkage of the top course. Sometimes the top course dishes up in a similar manner to the clay in the pond bottom.

In contrast to this phenomena is the manner in which gunite behaves. It is mixed relatively dry and is spread upon the surface it is to cover, in very thin layers. Each layer has less shrinkage to do because of its lack of free water and the shrinkage, if any, takes place relatively soon. If

microscopic cracks form, they are partially filled with the blast of the next infinitesimal layer and, if there is any shrinkage, it has little or no chance to find any distinct plane of any appreciable extent over which to distribute its effect. The result is a much sounder, denser and permanent surface.

Very few up-to-date cities now permit the placing of sidewalks or pavements in two courses of concrete, for the lower course is almost certain to be of a leaner mix—that is, less cement. With less cement there is less water and less shrinkage. In addition to that, whatever shrinkage takes place is generally over with by the time the second coat is applied. The second coat, being richer, contains more water, and this evaporates, causing a relatively great shrinkage. This in turn, makes the upper course crack or shear away from the lower.

Sometimes concrete is prevented from shrinking by the reinforcement, and hair cracks soon develop into structural cracks on the surface. This is particularly noticeable in ornamental objects made of concrete, such as bridge-spindles which have an hour glass shape, the crack being circumferential and coming at the small section.

Crazing cracks continue to grow with the age of the concrete, particularly when there are intermittent changes in the moisture content of the air or of the concrete itself. Their depth can be measured by means of spreading oil or red dye over the surface, and then sectioning the specimen.

Mr. Bates P. H. BATES.—It is possibly desirable to emphasize several points which have been but briefly covered in my paper or have not been mentioned at all. The first of these refers to the cement used. Before we started the investigation we secured a large shipment of clinker. This we have been grinding from time to time to the same fineness, and using in all the investigations. It may be that this cement is of a nature that mortars made from it will not readily craze. However, we did make up several panels using other cements which did not behave, however, any differently than did panels made from the cement prepared from the clinker. But it should be borne in mind that undoubtedly different cements will have different crazing tendencies.

At the bottom of the second page of my paper reference is made to a series of tests wherein immediately after making the panels the draft of an electric fan was passed over these until they had thoroughly hardened. The rapid evaporation produced by the blowing of the air over the wet specimens in no way whatever increased the tendency to craze. This was an unexpected result and seems to be contrary to what we have reported to us from the field. But it must be borne in mind that the wind blowing on stuccoed structures may be a hot dry wind or a hot humid wind, or a cold, dry wind or a cold, humid wind. Therefore a number of conditions may exist in the field, and the type of wind may produce decidedly different results.

In the first series of panels the mortars were placed upon 2 nonabsorbent bases, namely, a thin layer of rubber and a glass plate, in addi-

tion to placing upon a mortar base. These two distinctly different non-absorbent bases permitted of the mortar remaining plastic during the entire preparing of the specimen, and although a minimum amount of trowelling was used there was brought to the surface a certain amount of neat cement paste. When these specimens were later placed upon the roof they developed crazing very quickly, but if you will examine the majority of them now you will find the crazing has either disappeared or is rapidly disappearing. This is not a new phenomena at all but is one that has been commented upon repeatedly and is referred to in some cases as the sanding of the surface, or the ability of the stucco to keep itself clean. The thin film of cement paste under the action of the weather is disappearing through solution and with its disappearance we have the appearance of the sanded mortar. The crazing has been only in the thin film of paste. This is in contradiction to some of the statements that Professor White emphasizes, namely, that crazing is an indication of later failure of the concrete. Here we have the crazing disappearing with no evidence of failure of mortars and no evidence, even under magnification, of there being cracks in the mortars.

In the study of crazing the effect of workmanship must never be lost sight of. If a mortar is placed upon a base and but very lightly trowelled or worked, cracking or crazing may result in a very short time. In fact, generally cracking results along parallel lines, normal to the direction of trowelling, very quickly. This again has been noted by a number of people, particularly Mr. John J. Earley, and he has personally called my attention to the fact that thorough trowelling and compacting of stuccos or lime and gypsum plasters under the trowel will materially reduce the tendency to crack and craze. This has also been noted in some of our work.

On Fifth Avenue, New York City, a few blocks above 42nd Street, is a very excellent example of the cracking of stucco due to insufficient working. This is a church building of brownstone, and, as usual, the stone has spalled badly. The lower courses have been repaired by the characteristic brownstone stucco so much used in and around New York City. Due to the nature of the surface demanded, in repairing this spalled surface with the brownstone stucco thorough trowelling was not desirable. As a consequence there is the outstanding cracking.

W. K. Hatt.—In the engineering experimental station at Purdue University, we have been working for the past 4 years on matters relating to surface conditions of concrete—crazing, checking and cracking and questions of the ability of concrete to undergo vibratory stresses and volume changes. We hope to publish our material this summer. Mr. Hatt

We had rather large beams of concrete, some exposed in the air of the laboratory with a draft of hot air flowing on them all winter, and others subjected to contraction and expansion due to temperature and moisture changes in the outdoor air. Just as Mr. Bates found, the beams which were in the best condition were those that had been subjected to the blowing of hot air, i. e., a uniform condition rather than the alternating condition of the outside air.

We have tried to distinguish between so-called cracks and water marks and crazing. When people talk about cracks, they are not always talking about the same thing, and we have tried to define the differences between so-called crazes and small fissures which appear as water marks and structural cracks which occur in reinforced concrete when the steel has exceeded its elastic limit. We have defined these differences by the width of the crack.

We have worked on neat cement and also on concrete. We find about as Professor White found in his investigation that the neat cement beams (during the past 3½ yr.) have grown in length, whereas that concrete in the same time has diminished in length with the alternating dryness and wetness. I think it is quite a long jump to search out the quantitative conditions of neat cement and apply them to concrete. To predicate failure upon the appearance of surface checks in deep cement beams of this size seems to me an unwarrantable conclusion.

We have also gone ahead with reference to what happens to concrete under exposure. A good many crazes which we found on beams stored in the outside air were only in the surface film of the cement; they did not go down into the body of the concrete as fissures separating the cement from the sand. We also broke these beams in the laboratory to determine if these so-called surface fissures would produce a structural crack when the beam failed. In very few cases did they do so.

Mr. Toch. MAXIMILIAN TOCH.—I had an idea that Professor White or Mr. Bates or Professor Hatt were going to discuss this subject. I mention this to demonstrate that while I did not know what was in the other papers I arrived at practically the same conclusions as the others. Our laboratory for the past year has been conducting a large number of practical experiments in order to determine, if possible, what the cause of crazing was and what remedy could be applied.

The crazing of portland cement concrete is a problem that has disturbed engineers and contractors for many years, and it is always apparent either on the surface or on a side wall. Sometimes the blame is put on the cement and sometimes on the aggregate, but up to now the actual cause has probably never been thoroughly studied.

The appearance of crazed cement may be either very fine hairline intercepting cracks or it may manifest itself in heavier fissures. From the studies made under my direction at our laboratory, it is quite apparent that the whole trouble is due to a physical reaction known as "surface tension."

In order to understand what "surface tension" is, it is best to explain this in simple language. For hundreds of years artistic painters have been troubled with the peculiar reaction that the fresh paint would not take hold of the undercoat but, after a few minutes, would begin to curl up and coalesce. Many mediums have been invented to overcome this particular effect, some of which have been quite successful. This inherent characteristic of fresh oil paint to form little drops and streaks over a freshly painted

surface, is due to what chemists and engineers know as "surface tension"; that is, the inherent ability of a liquid to form drops. Another more common example is that of rain on a window-pane. The drops do not flow together and form one even sheet, but each drop has in it the inherent ability to coalesce or remain together until another drop is added and then by sheer force of gravity it flows down; but the window-pane is not uniformly wet.

Water is known to have a very great surface tension; in fact, it has a greater surface tension than many other liquids. If instead of water we sprinkle either alcohol or turpentine on a window-pane, drops would not form, but the liquid would wet the plate evenly. This is because materials like alcohol, turpentine and other volatile solvents have a low surface tension with an ability to combine and spread and not to coalesce.

Following out that theory, a number of experiments were made with concrete, beginning with 6½ gal. of water to the bag of cement and going up to 9 gal., with surprisingly uniform results. My attention was first called to this peculiar reaction after the Army and Navy Building at Washington had been constructed during the war, with feverish haste and in record time. This building is about 1,000 ft. long, entirely built of concrete, and the floors throughout the entire building had no cracks in the ordinary sense, but heavy fissures developed. In order to cast quickly and finish the job, an excess amount of water was used with the result that before a month crazing appeared and many of the floors have since been covered with linoleum in order to make them level.

The inherent ability of water to contract within itself is the cause of crazing. It has been stated by many that 7 gal. of water is the maximum amount to be used to a bag of cement. I find that 6½ gal. would produce much better results. But in my operations, 6½ gal. of water, although sufficient to produce a very strong concrete, does not flow out of the mixer, and the fallibility of human nature is such that the man at the mixer will dump in extra water.

This leads me to the question as to whether concrete is permanent or not. It is perfectly true that concrete has only been a stabilized material for perhaps 30 years, although it has been in use 75 years. This may be correct as far as the portland cement is concerned, but it must be borne in mind that cementitious materials which liberate lime have been in use for thousands of years and have been absolutely permanent. We have two gigantic structures, the Chinese Wall and the Roman Aqueduct which should give us the assurance that a lime bearing cementitious material will stand. Wherever concrete has failed it has been more or less due to extraneous causes, such as poor mixing, too much water and impure aggregates.

I cannot agree that the present form of rotary mixer is by any means the best apparatus for mixing concrete. Now that we know more about portland cement and its hydration, it is time that we invent a mixing machine which will dump its contents when the minimum amount of water

is used. After all, there is no great difference between the mixing of concrete and the mixing of heavy paste paints, or bakers' dough. Neither the paint manufacturer nor the baker uses the type of mixer which is used for making concrete. The present type of mixer is an incentive to add more water than is necessary.

In conclusion, it is very evident to me, that crazing is due to a sloppy mix, the excessive water on the surface having a tendency to coalesce from the surface tension and on drying out producing the crazing. This does not happen when the mix is fairly dry and properly troweled.

Member. MEMBER.—There seems to be a discrepancy between the results obtained by Professor White and by Mr. Bates. Mr. Bates suggests that the discrepancy might be due to differences between the different brands of cement. A slide shown by Mr. Hatt shows a large discrepancy between the different brands. In comparing the results, it seems there is a wide discrepancy in the drying out of the specimens. I assume that in the drying of Professor White's specimens there was a very slow drying corresponding to what we would have on the job where the surface is dry and the main bulk of the material wet, so that the main bulk is expanded and the surface is in tension. Now a very slow process of drying may mean that the main bulk of the material dries out so slowly that the tension on the surface is not able to produce a crack. I think the gentleman should iron out the discrepancy between the experiments.

Mr. Bates. P. H. BATES.—I interpreted Professor Hatt's experiments to show that there was a very marked difference between the brands of cement. Is that true, Doctor?

Mr. Hatt. W. K. HATT.—Yes.

Mr. Davis. R. E. DAVIS.—I think all of us who have made anything of a study of volumetric changes in concrete appreciate that this is a very important subject. We understand that these volumetric changes must be constantly going on unless a concrete or mortar is immersed in water for a long period of time. In the materials testing laboratory of the University of California, we have been carrying on a series of investigations along these same lines during the past four years. Our specimens have been concrete, cement mortars and neat cement. We have gone to considerable pains to remove certain variables. We have stored the specimens under constant conditions as regards humidity and temperature, having prepared special rooms in which to house our pets. At the present time we have something like 300 under observation. Without going into the details of these tests, I would like to call attention to the fact that, other things being equal, the difference in shrinkage depends upon atmospheric conditions. These specimens have been allowed to shrink under humidities of 50, 75 and 95 per cent. We find that the humidity has a marked influence, both on the shrinkage and on the rate at which they expand after they have been dried and placed in rooms of a given humidity.

One very interesting series of tests was started last summer on the behavior of mortar in brick masonry. These tests have now been in prog-

ress during a period of six months. The same cement mortars have been cast in the form of bars in various types of molds and have also been used in brick pier construction. Those cast in waterproof molds and then removed from the molds have shrunk more rapidly than any of the others, and of course are continuing to shrink. Those cast in terra cotta molds and then removed from the molds at the age of two days, have likewise shrunk from the beginning, but at a less rapid rate. Those cast in terra cotta molds and left within the molds swelled for a period of time, about a month on the average, and then began to shrink, and are still shrinking. The mortars used in brick piers have been expanding during the six months and are still expanding, although at a much less rapid rate; doubtless they will soon come to rest and will then begin to shrink.

Three sides of these piers are kept away from the air by means of waterproof building paper; the third side is exposed to the air. In opposite sides are strain gauge plugs so that the shrinkage of the side exposed to the air or the change in length of the side away from the air can be determined. The side next to the air, while it swelled for about two months, is now shrinking, but the side not exposed to the air is still expanding at a fairly rapid rate. I think this behavior would substantiate what both Professor White and Mr. Toch have said. It may be due either to chemical or to physical causes.

C. M. Chapman.—We frequently talk about the shrinkage and expansion of concrete caused by changes in humidity, and about the effect of varying quantities of cement, or richness of the mix, upon volume changes caused by moisture, but little or nothing has been said here about the effect of the aggregate. **Mr. Chapman.**

Perhaps you have taken the aggregate into consideration, but if not, a very important element has been overlooked. The character of the fine aggregate has a very marked influence upon volume changes.

A series of tests made in the laboratory of Dwight P. Robinson & Co., in 1923, gave some striking results in this respect.

Mortar made with certain aggregate showed volume changes due to moisture of as much as four and a half times the volume change of mortar made with other aggregates. These tests were reported before the A. S. T. M. in 1924 and are recorded in their proceedings for that year.

Study of a Method for Testing Concrete in the Field.

By C. A. Wiepking.*

In 1926 Messrs. Clemmer and Burggraf published the results of their series of tests investigating the possibilities of using beam specimens for field tests on concrete.[1] The cantilever beam test was proposed to replace the test on the standard 6 x 12-in. concrete cylinder. The intention was to avoid the inconvenience entailed in sending concrete cylinders to the testing laboratory, and also the variations that commonly occur in compression tests.

It was appreciated that a saving of time and expense, as well as better application to field control of concrete, would be accomplished by adopting the beam test. The transverse test has been used in a number of places, and several highway organizations have employed it extensively for determining the quality of concrete or the date for opening pavements to traffic. Several papers have been published giving the results of field experiences with the new method of testing.

From the published articles concerning the transverse test, it appears that the size of beam generally used is 6 x 8 x 30 in. Specimens having a cross section of 6 x 6 in. have also been used. The length of these test pieces could be reduced to 20 in., and one transverse test could then be made on each specimen. However, it has been found desirable to make two tests on each beam, for which a length of about 30 in. is necessary.

The advantages of the transverse test are in the simple apparatus, the low test loads, and the fact that the testing can be done on the job, where the results may find immediate application. The fact that the men on the job can see the test performed and thus better appreciate the quality of the concrete, is of considerable importance.

Considering the scheme in greater detail some unhandy features become apparent. If a 6 x 8-in. concrete beam is to be broken at the 28-day age by means of a cantilever, the test load will be fairly large, if the concrete is of good quality. Suppose, for example, that the concrete is to have a compressive strength of 2,500 lb. per sq. in. at 28 days. The modulus of rupture or beam strength is roughly one-fifth as large a number, or 500 lb. per sq. in. A 6 x 8-in. beam laid flatwise (8-in. sides horizontal) has a "section factor" of 48 in.^{3}, and the moment required to break the concrete is therefore 24,000 in.-lb. This requires a load of 333 lb. at the end of a 6-ft. cantilever, or 250 lb. for an 8-ft. cantilever, or 200 lb. for a 10-ft. cantilever. Testing such beams cannot be regarded as a simple job, because the load to be handled is too large, or the lever too long, for convenience.

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[1] *Proceedings* of A. C. I., Vol. XXII, 1926, p. 304.

Long levers must be heavy, and therefore cumbersome, in order to be sufficiently rigid.

Of course the test would be much easier to make at the 7-day age, because the strength developed by the concrete at that time is less. No difficulty will be encountered in the testing of weak mixes of concrete, even if the large beam specimen is used.

The present-day trend is toward quality concrete, and a compressive strength of 2,500 lb. per sq. in. is not a high value. Mixes developing a strength of 3,000 lb. per sq. in. and more, at 28 days, are not uncommon. To test such concrete by means of a lever might become a difficult procedure.

One of the most practical applications for a field testing scheme would be in determining the strength of high early-strength concrete. Such concrete tested at ages of two to four days, would easily carry loads larger than those mentioned above.

The weight of the 6 x 8 x 30-in. specimen is in itself a disadvantage. A concrete beam of these dimensions weighs just about 125 lb. On road jobs where the test can be made by clamping the specimen to the back of a truck, there may be no inconvenience in handling such test pieces. However, most other construction jobs afford little space for making and testing these specimens, and it may not be convenient to handle test pieces of such weight.

These considerations led to the question of whether a smaller test specimen might not be effectively used in the transverse test. It seems desirable to have a specimen which one man can conveniently carry from any part of the construction site to the location of the testing apparatus. Accordingly the 4 x 6 x 24-in. specimen was suggested, because such beams weigh about 50 lb. each, and the estimated testing loads are small enough. Due to the smaller size of cross-section, a length of 24 in. is sufficient to allow for two tests per specimen.

The larger sizes of transverse test specimens should give valid results because a relatively large sample of concrete is contained in each. Choosing a smaller specimen involves the question of reliability of results, because a smaller sample of concrete will be used in making each beam, and because a 4-in. width of mold might cause difficulties in puddling the concrete. The tests reported herein were made to investigate, as far as possible, these questions involving the two sizes of beams. The ultimate aim in this work is to develop an acceptable and reliable procedure which can be applied on any concrete construction job without expensive apparatus, and with only reasonable effort.

PROGRAM OF TESTS.

The series of mixes for this investigation was planned primarily to give a large number of batches of workable concrete of average richness, so that the uniformity of transverse test results on the 6 x 8 x 30-in. and the new 4 x 6 x 24-in. beams could be compared.

Since the compressive strength has long been recognized as an im-

portant index of quality of concrete, the program called for compression tests on 6 x 12-in. cylinders. The ratio of modulus of rupture to compressive strength was to be found for each size of beam.

Provision was made for gravel and crushed stone as coarse aggregates, and for dry, medium and wet consistencies. One oversanded mix and one undersanded or harsh mix were included. Batches with different sizes of coarse aggregate were planned, because of possible differences in puddling the concrete into the molds. Trap rock, red granite, Oshkosh limestone, Madison sand and limestone screenings were designated for certain batches, because of the individual characteristics of these materials.

A portion of the program was duplicated with 3-min. mixing instead of 1½ min.; this did not lead to much difference in results, but it did add to the number of values used as a basis for comparison of tests. Enough beams and cylinders were made for 7-day, 28-day, and one-year tests; these specimens were cured under ordinary weather conditions. Additional pieces were made in a part of the program for 7-day and 28-day damp-sand curing.

Three specimens of each kind were made for each variable at each test age. A one-bag batch produced enough concrete for three cylinders and three of each size of beam. The specimens were marked according to a numbering scheme that spread the pieces from each batch among the several test ages.

The schedule called for two tests on each beam. Accordingly each beam was inverted before the second test, to place the tensile stress on the opposite side from the original. Averaging the two results could be expected to give more reliable values.

BEAM TESTING APPARATUS.

The apparatus for testing the beams was designed very nearly like the outfit described by Messrs. Clemmer and Burggraf in the paper presented at the 1926 convention of the American Concrete Institute. A blueprint plan of the testing beam used by the Illinois Division of Highways was consulted to get the desired dimensions. Fig. 1 is a diagram showing the arrangement that was actually used.

The rigid base or bench for clamping specimens was made by pouring a rich mix of concrete into a rectangular form built against a gravel bin. The resulting block of concrete was about 4 ft. long, 14 in. wide, and 4 ft. high. This made a bench of suitable height and heavy enough to resist overturning. The bolts for clamping specimens were anchored in the concrete by means of steel plates bedded about 10 in. deep. A steel angle was bedded flush at the front of the bench to protect the concrete from wear and to act as a bearing edge over which the concrete would break.

The first testing lever was 6 ft. long, as shown in the diagram. After the 28-day tests on the 6 x 8 x 30-in. beams came due, the loads were so large that progress in testing was slow and difficulties were encountered

in handling the required amount of steel shot and sand. A 12-ft. lever was built for use on the stronger specimens. The greater leverage reduced the load, but the new lever required more care in attaching it to the test beams. When the 1-year tests were being made the 12-ft. testing lever was barely adequate, as it deflected considerably under the high loads required to break the concrete.

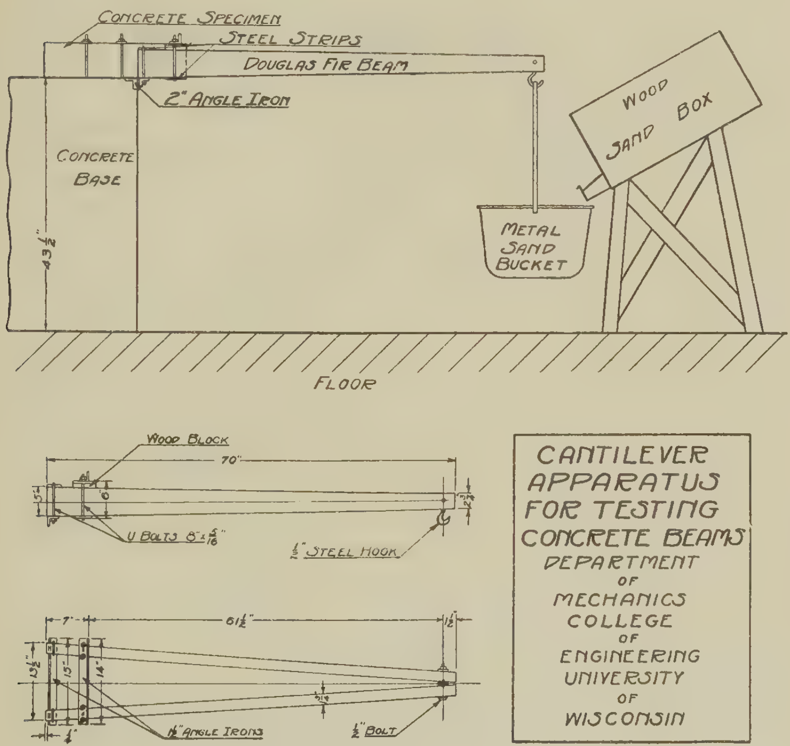

FIG. 1.—BEAM TESTING APPARATUS.

MATERIALS.

Cement.—The cement used in these tests was a mixture of three standard brands: Universal, Medusa, and Lehigh. Shipments of 40 sacks of each brand were obtained in the local market and the mixture was made by dumping from the different brands alternately one sack at a time into a tight galvanized iron bin. After six bags had been dumped into the bin the mass was raked to spread and mix it, and so successive layers were added and raked. Due to storing in the galvanized iron bin the cement remained fresh throughout the series of tests. Portions were drawn through a spout at the bottom of the bin as needed.

Each of the three shipments of cement was sampled before mixing, and the samples were submitted to the standard tests. It was found that each of the three brands met the standard requirements. A composite sample was taken from the mixed cement and this was also tested in order to furnish a check on the quality.

Fine Aggregates.—Janesville torpedo sand from the local market was used in most of the mixes of concrete in this series. A sufficient supply for the entire series was kept in a separate bin, and samples were taken from time to time to determine the fineness modulus and check the uniformity. Janesville sand is a well-graded fine aggregate; it is a typical southern Wisconsin sand.

Madison sand, which was obtained from the pit on the university grounds, was used in only one mix of concrete. This is a rather fine sand,

TABLE 1.—SIEVE ANALYSES OF FINE AGGREGATES.

Sieve No.	Per Cent, by Weight, Retained on Each Sieve		
	Janesville Sand	Madison Sand	Limestone Screenings
4	3.0	0	9.8
8	15.6	0	34.3
14	25.8	4.9	53.1
28	39.0	13.3	62.4
48	80.4	56.2	70.4
100	96.4	90.1	81.0
Total	260.2	164.5	311.0
Fineness Modulus	2.6	1.6	3.1

not suitable for use in important concrete work. It contains practically no particles larger than the 14-mesh size.

The limestone screenings used in two of the mixes were obtained from the Lutz Stone Company, of Oshkosh, Wisconsin. This material was received in cement sacks and was used in the batches just as furnished by the above named firm. A considerable portion of the bulk in these screenings consisted of crusher dust, and the larger chips were about ¼ in. in size.

Table 1 gives the sieve analysis of each of the fine aggregates.

Coarse Aggregates.—Janesville gravel was used in a number of the mixes of concrete. The supply was obtained in the local market, and it was screened out to three sizes and recombined as desired. In most of the mixes the three sizes were proportioned to produce an almost ideal grading. In some of the batches gravel of uniform size was used. Table 2 gives the grading of the different gravel combinations. Janesville gravel consists largely of dolomite. Most of the particles are round, but a small proportion of fractured material is included.

The Lannon stone was obtained from a Madison contractor who had a large quantity of this material on hand for a construction job. Because this supply contained very small proportions of smaller sizes it was decided to use uniform coarse material in all except one of the Lannon stone

TABLE 2.—TYPES OF AGGREGATES; GRADING, WEIGHT PER CUBIC FOOT.

SCREEN SIZES
No. 1 Size is ¼-in. round to ⅜-in. mesh.
No. 2 Size is ⅜ to ¾ in.
No. 3 Size is ¾ to 1½ in.

Name of Aggregate	Source	Grading	Fineness Modulus	Weight per Cubic Foot, lb.	Used in Mixes No.
FINE AGGREGATES					
Janesville Sand	Janesville S. & G. Co., Janesville, Wis.	0 to No. 4	2.6	115	1 to 14 incl., 16–20, 23
Madison Sand	University Pit	0 to No. 4	1.6	105	15
Limestone Screenings	Lutz Stone Co., Oshkosh, Wis.	0 to ¼ in.	3.1	120	21, 22
COARSE AGGREGATES					
Janesville Gravel	Janesville S. & G. Co., Janesville, Wis.	1 part No. 1 2 parts No. 2 3 parts No. 3	7.33	105	1 to 6 incl., 16, 17, 18
Janesville Gravel	Janesville S. & G. Co., Janesville, Wis.	No. 4 to ⅜ in.	6.0	100	11
Janesville Gravel	Janesville S. & G. Co., Janesville, Wis.	⅜ to ¾ in.	7.0	100	12
Janesville Gravel	Janesville S. & G. Co., Janesville, Wis.	¾ to 1½ in.	8.0	100	13, 15
Lannon Stone	Lake Shore Stone Co., Lannon, Wis.	¾ to 1½ in.	8.0	96	7 to 10 incl.
Lannon Stone	Lake Shore Stone Co., Lannon, Wis.	1 part No. 1 2½ parts No. 2 2½ parts No. 3	7.26	100	21
Red Granite	Wisconsin Granite Co., Red Granite, Wis.	1 part No. 1 2 parts No. 2 3 parts No. 3	7.33	105	19
Trap Rock	Trap Rock Co., Dresser Junction, Wis.	1 part No. 1 3 parts No. 2 6 parts No. 3	7.50	105	23
Limestone	Lutz Stone Co., Oshkosh, Wis.	1 part ¼ to ½ in. 2 parts ¾ to 1¼ in. 2 parts 1½ to 1¾ in.	8.0	100	20, 22

mixes. The Lannon material is a crushed grey limestone and is regarded as being of good strength and durability.

The Red Granite used in mix number 19 was obtained from the Wisconsin Granite Company, from the quarry at Berlin, Wisconsin. The ship-

ment of this material contained a range of sizes, so that it was possible to screen the material and recombine it according to an arbitrary gradation. The granite was incorporated into the program because it is a hard strong aggregate and its surface characteristics might affect the results of transverse tests. Crushed limestone was obtained from the Lutz Stone Company of Oshkosh, Wisconsin. The shipment consisted of three different sizes of material, in cement sacks, and therefore it was used in the batches according to a definite ratio of these three sizes. Trap rock for mix number 23 was obtained from the quarry of the Trap Rock Company at Dresser Junction, Wisconsin. This material was also available in several sizes so that it could be screened and recombined.

The above materials were selected for these tests because they are typical concrete aggregates in use in different sections in the state of Wisconsin. At the same time the characteristics of these materials are not alike and therefore a number of different conditions are brought into the mixes.

Table 2 gives the weights per cubic foot of all the aggregates as well as the fineness moduli, size ranges, and numbers of the mixes in which they were used.

PREPARATION OF MATERIALS.

All of the aggregates were used in the air dry condition. The materials were spread on the laboratory floor and occasionally raked, until all the excess moisture had dried out. Weight per cubic foot tests were made on the dry materials for use in computing batch quantities, and samples were taken for the sieve analysis.

The Janesville sand was passed through a ¼-in. round screen (equivalent to No. 4) in order to remove the larger pebbles, which were then added to the gravel. By this method a uniform well-graded sand ranging from a small proportion of dust up to the number four size was obtained. The coarse aggregates, except the limestone from Lutz Stone Company, were screened through a sieve that separated the material into three sizes, as listed at the top of Table 2. Material passing over the 1½-in. screen was generally discarded. The three sizes of each aggregate were kept in separate bins until the time for proportioning the batch. Proper amounts of each size were weighed out as required. This procedure was followed to prevent variation due to the segregation that occurs when all sizes are together in a single bin or pile.

PROPORTIONING.

The mixes in this series were generally proportioned 1:2:4 by dry volume. Knowing the weights per cubic foot of the dry materials, the desired quantities of sand and coarse aggregate were computed in pounds and the batches were weighed out accordingly. The water was weighed out in a bucket on a platform scale.

MIXING.

The concrete was mixed in a No. 0 Smith mixer run at a speed of 28 to 30 r. p. m. The inside of the mixer was wetted and allowed to drain for about a minute before the first batch of each day. This precaution was taken so that the interior of the mixer would not have to be wetted by part of the mixing water from the batch. The mixer was flushed with water and drained between batches.

The dry batch of material was run from the scale hopper cart directly into the mixer and the water was added through the charging spout at the end of one-half minute of dry mixing. After the addition of the mixing water, the mixer was run for a minute longer, or for two and one-half minutes, according to the total time required by the mixing program. At the end of the mixing period the batch was dumped into two wheelbarrows and taken out to the molding platform.

MOLDING THE SPECIMENS.

The beam and cylinder specimens were molded on a level platform outside of the mixing shed. The beam specimens were made in steel molds which consisted of channel irons bolted together. Wooden separators were used in the channel molds so that beams of 24 and 30 in. length could be made. The beam molds were set up 6 in. wide by 8 in. deep for the large beams, and 4 in. wide by 6 in. deep for the small specimens. The cylinders were cast in 6 x 12-in. steel pipe molds which had been set upon a level steel plate. All molds were properly clamped and oiled before any mixing was begun.

The two wheelbarrows of material from the mixer were dumped upon a level platform which had been wetted to prevent absorption. The edges of the pile of concrete were shoveled to the middle in order to mix the batch. Portions were taken from different parts of the batch and poured into the two kinds of beam molds and the cylinder molds alternately so that all parts of the batch would be represented in each of the specimens.

The cylinders were rodded in the standard manner using a ½-in. steel rod. The beam specimens were compacted by pushing a flat shovel down into the channel molds on each side, and by tapping the top surfaces with the edge of a trowel. The large and the small beams were molded in the identical manner. All of the specimens were troweled smooth after the concrete had settled for a short time.

CURING.

The specimens were removed from the molds on the day after pouring. Each specimen was marked with the mix number and then an individual number. The numbers were so arranged that the three specimens of a kind from any one batch were designated for testing at different ages. Some of the specimens were packed in damp sand immediately after re-

moval from the molds, and they were kept under sand until the seven or twenty-eight day test age. The sand pile was sprinkled every morning so that the sand surrounding the specimens would remain damp at all times.

The specimens for dry curing were set in rows on strips of wood which had been laid on the ground. These specimens were not protected in any way, but they were raised off of the ground about 2 in. by the wooden strips in order to allow circulation of air under them and to prevent absorption of moisture from the earth.

TESTING.

At the proper testing age the specimens were moved into the mixing shed and weighed. The 6 x 12-in. cylinders were measured for diameter and length and they were capped with plaster of paris to provide smooth flat ends for contact with the bearing plates of the testing machine. The cylinders were tested under a spherical head and the usual observations of maximum load and character of fracture were noted.

The beam specimens were tested by means of the lever apparatus shown in Fig. 1. Each beam was bolted to the fixed support and width and depth were measured at the portion of the beam next to the bearing edge. The wooden testing lever was then fixed on to the specimen as shown in the figure, and the load of sand in the bucket was gradually increased by allowing sand to flow down from the hopper. When the specimen broke the flow of sand was stopped immediately and the bucket of sand was weighed on a platform scale. The specimen was unbolted and turned end over end; it was again bolted to the bench so that a second test could be made. In this second test the side of the beam which had been in compression was now the tension or upper side. The wooden testing lever was again attached and the new breaking load was determined.

The 4 x 6-in. beams were tested with the 6-in. faces vertical, or in the same position as they were molded. Thus the troweled surface was up in the first test on the beam and down in the second test. The 6 x 8-in. beams were tested on flat faces, or with 8-in. faces horizontal. Thus the sides in molding were top and bottom in testing. After every break the broken concrete was examined to note what proportion of the aggregate had been fractured in the tensile and in the compression part of the beam.

One short end piece of every beam was saved for a compression test. In case of slant or irregular fracture the broken end was chiselled approximately square. Each end piece was capped in plaster of paris on both faces and tested in the usual manner. The load was applied to the faces that had been top and bottom in the mold.

The information obtained on the beam ends was so irregular that no attempt has been made to exhibit the results in this report. The compressive strength values for the beam ends were in most cases considerably higher than for the corresponding cylinders; this was due to the low ratio of height to width, as compared to the corresponding ratio for the cylinders.

COMPUTATIONS.

The unit compressive strength was computed for each 6 x 12-in. cylinder. The maximum compression load was divided by the actual cross-

TABLE 3.—RESULTS OF TRANSVERSE AND COMPRESSIVE TESTS.

Materials: Mixed Cement; Janesville Sand; Janesville Gravel.
Mix: 1:2:4 by dry volume.
Curing: Outdoors, under ordinary weather conditions (August); some specimens protected under damp sand, as noted.
Each strength value in table represents average of results of tests on three specimens.

Mix No.	Time of Mixing, minutes	Slump, in.	W/C	Curing	Age	Modulus of Rupture, lb. per sq. in.		Compressive Strength, lb. per sq. in. S	Strength Ratios	
						4 x 6 Beam R_1	6 x 8 Beam R_2		R_1/S	R_2/S
1	1½	¼–½	0.79	dry	7 days	474.2	512.8	2971	0.160	0.173
				dry	28 days	555.7	584.4	3404	0.163	0.172
				dry	1 year	799.1	627.3	5480	0.146	0.115
				damp sand	7 days	471.2	520.7	3046	0.155	0.171
				damp sand	28 days	646.0	694.0	4180	0.154	0.166
2	1½	3–4	0.87	dry	7 days	451.4	502.8	2514	0.179	0.200
				dry	28 days	527.3	525.7	2882	0.183	0.182
				dry	1 year	784.2	610.0	4120	0.190	0.148
				damp sand	7 days	451.2	465.8	2696	0.168	0.173
				damp sand	28 days	603.2	614.0	3757	0.160	0.163
3	1½	8–9	0.99	dry	7 days	370.7	441.6	2348	0.158	0.188
				dry	28 days	435.6	457.7	2000	0.218	0.229
				dry	1 year	676.8	517.3	4106	0.165	0.126
				damp sand	7 days	446.6	391.4	1895	0.236	0.206
				damp sand	28 days	577.1	506.4	2800	0.206	0.181
4	3	¼–½	0.79	dry	7 days	483.7	553.4	2560	0.189	0.216
				dry	28 days	514.7	574.3	3444	0.149	0.166
				dry	1 year	803.3	618.0	4553	0.176	0.135
				damp sand	7 days	484.3	497.3	2933	0.165	0.169
				damp sand	28 days	609.5	671.0	4337	0.140	0.155
5	3	3–4	0.87	dry	7 days	391.0	473.0	2206	0.177	0.214
				dry	28 days	505.4	566.2	2714	0.186	0.208
				dry	1 year	608.6	647.5	4860	0.125	0.133
				damp sand	7 days	460.2	454.6	2835	0.162	0.160
				damp sand	28 days	625.4	598.2	4066	0.154	0.147
6	3	8–9	0.99	dry	7 days	365.2	378.7	1864	0.195	0.203
				dry	28 days	450.0	476.0	2247	0.200	0.212
				dry	1 year	649.3	569.3	3980	0.163	0.143
				damp sand	7 days	437.6	453.4	2442	0.179	0.185
				damp sand	28 days	543.0	542.8	3181	0.171	0.171
Average Values									0.172	0.174

section area of the cylinder; because of the slight variation in diameter the area of each cylinder had been determined. Average compressive strength values were computed for each set of three cylinders of a kind at each age; these average values are given in Tables 3 to 6 under the symbol "S."

The modulus of rupture was computed from each beam test. The maximum or breaking load in each case was multiplied by the lever arm, which was the distance from the loading bucket hook to the steel edge of the testing bench. To the resulting moment was added the correction factor for dead weight of the testing lever, namely weight of lever times distance of its center of gravity from the edge of the bench.

TABLE 4.—RESULTS OF TRANSVERSE AND COMPRESSIVE TESTS.

Materials: Mixed Cement; Janesville Sand; Lannon Stone.
Mix: 1:2:4 by dry volume.
Curing: Outdoors, under ordinary weather conditions (September); some specimens protected under damp sand, as noted.
Each strength value in table represents average of results of tests on three specimens.

Mix No.	Time of Mixing, minutes	Slump, in.	W/C	Curing	Age	Modulus of Rupture, lb. per sq. in.		Compressive Strength, lb. per sq. in. S	Strength Ratios	
						4 x 6 Beam R_1	6 x 8 Beam R_2		R_1/S	R_2/S
7	1½	3–4	0.87	dry	7 days	349.1	300.0	1618	0.216	0.185
				dry	28 days	502.3	464.8	2192	0.229	0.212
				dry	1 year	737.2	772.3	3323	0.222	0.232
				damp sand	7 days	410.2	399.4	1723	0.238	0.231
					28 days	619.2	495.1	2607	0.237	0.190
8	1½	8–9	0.99	dry	7 days	273.8	267.3	1206	0.226	0.222
				dry	28 days	379.4	394.6	1651	0.230	0.239
				dry	1 year	671.3	638.0	2542	0.264	0.251
				damp sand	7 days	259.7	269.0	1099	0.236	0.244
					28 days	582.0	459.7	2258	0.257	0.203
9	3	3–4	0.87	dry	7 days	349.6	374.2	1553	0.225	0.241
				dry	28 days	400.6	459.3	1887	0.212	0.243
				dry	1 year	777.3	786.3	3090	0.251	0.254
				damp sand	7 days	321.4	388.6	1563	0.206	0.248
					28 days	616.2	561.8	2817	0.219	0.200
10	3	8–9	0.99	dry	7 days	276.5	308.0	1338	0.207	0.230
				dry	28 days	424.0	396.1	1778	0.238	0.223
				dry	1 year	659.7	690.0	2690	0.245	0.256
				damp sand	7 days	302.9	309.2	1231	0.246	0.251
					28 days	444.7	456.7	2035	0.218	0.224
Average Values									0.231	0.229

The total breaking moment in inch-pounds as computed above was divided by the "section factor" $\frac{bd^2}{6}$ for the rectangular beam cross-sections, for each test. Actual measurements of b and d at the breaking section were used in these computations, because the cross-sections of the specimens had some variation. The moduli of rupture computed in this manner were designated R_1 for the 4 x 6 x 24-in. specimens and R_2 for the 6 x 8 x 30-in. specimens. The two computed values for each beam were aver-

aged to give a single value representing the strength of the specimen. Results were grouped for each set of three specimens of a kind. Thus the values in columns R_1 and R_2 in Tables 3 to 6 are averages for three beams or six tests each. Ratios of moduli of rupture (or transverse strength) to compressive strength were computed as recorded in the tables under

TABLE 5.—RESULTS OF TRANSVERSE AND COMPRESSIVE TESTS, CONCRETE WITH VARIABLE GRADING OF AGGREGATE.

Materials: Mixed Cement:
Janesville Sand (J) or Madison Sand (M).
Janesville Gravel (J) or Lannon Stone (L).
Mixing: All batches mixed ½ min. dry and 1 min. wet, for a total of 1½ min., in a No. 0 Smith mixer.
Curing: In molds one day; then placed on strips of wood and subjected to ordinary weather conditions. (August-September).
Each strength value in table represents average of results of tests on three specimens.

Mix No.	Materials: Sand	Materials: Coarse Aggregate	Grading of Coarse Aggregate	Mix Ratio by Weight	Fineness Modulus	Slump, in.	W/C	Age	Modulus of Rupture, lb. per sq. in.: 4 x 6 Beam R_1	Modulus of Rupture, lb. per sq. in.: 6 x 8 Beam R_2	Compressive Strength, lb. per sq. in. S	Strength Ratios: R_1/S	Strength Ratios: R_2/S
11	J	J	No. 4 to ⅜ in.	1:2.3:4.0	4.76	2–3	0.94	7 days	396.1	403.2	1863	0.212	0.216
								28 days	477.0	577.5	2840	0.168	0.203
								1 year	897.5	771.8	5220	0.172	0.148
12	J	J	⅜ to ¾ in.	1:2.3:4.0	5.40	3–4	0.94	7 days	347.3	373.1	1914	0.181	0.195
								28 days	468.3	576.0	2938	0.159	0.196
								1 year	840.8	687.2	5250	0.160	0.131
13	J	J	¾ to 1½ in.	1:2.3:4.0	6.03	3–4	0.94	7 days	366.5	343.7	1391	0.263	0.246
								28 days	492.7	482.6	2559	0.193	0.189
								1 year	653.5	585.7	3548	0.184	0.165
14	J	L	¾ to 1½ in.	1:2.3:4.0	6.03	3–4	0.94	7 days	324.4	388.8	1493	0.217	0.260
								28 days	422.1	516.2	2479	0.170	0.208
								1 year	656.0	620.6	3502	0.187	0.177
15	M	J	¾ to 1½ in.	1:2.1:4.0	5.66	9	0.92	7 days	296.7	307.7	1401	0.212	0.219
								28 days	338.1	396.9	1867	0.181	0.212
								1 year	559.3	612.2	2920	0.191	0.210
16	J	J	No. 4 to 1½ in.	1:3.76:2.74	4.59	¼	0.87	7 days	344.0	420.9	1432	0.240	0.294
								28 days	435.7	464.2	2565	0.170	0.181
								1 year	695.2	645.6	4340	0.160	0.149
17	J	J	No. 4 to 1½ in.	1:3.07:3.43	5.10	6	0.87	7 days	416.8	432.2	1970	0.211	0.219
								28 days	524.9	516.8	2642	0.198	0.195
								1 year	662.2	700.6	5166	0.128	0.135
18	J	J	No. 4 to 1½ in.	1:1.15:4.95	6.20	9	0.87	7 days	262.9	338.7	1383	0.190	0.245
								28 days	447.9	477.2	1842	0.243	0.259
								1 year	700.5	673.7	3310	0.211	0.205

"Strength Ratios." These values were found directly from the average values in the tables, as indicated by R_1/S and R_2/S. The ratios thus compare the results of tests on the 4 x 6 x 24-in. beams and on the 6 x 8 x 30-in. beams, with the results of tests on the 6 x 12-in. cylinders.

DISCUSSION OF CURVES.

Moduli of Rupture vs. Compressive Strength.—In Figs. 2 to 5 are plotted the strengths of all the 4 x 6-in. beams and 6 x 8-in. beams, against

the strengths of the corresponding compression cylinders. Because of the diversion of results it was found desirable to plot the values for the gravel concretes on the first two graphs and the values from the crushed stone concrete on the next two.

The dash line through the group of plotted points in Fig. 2 represents the average ratio 0.172 of R_1/S from Table 3. According to this ratio the modulus of rupture for a 4 x 6-in. beam is about 17 per cent of the compressive strength of the concrete. It will be noted that the plotted points follow the line fairly well. For low strength values the

TABLE 6.—RESULTS OF TRANSVERSE AND COMPRESSIVE TESTS; CONCRETE MADE WITH VARIOUS AGGREGATES.

Cement: Mixture of 3 brands. Aggregates as noted below.
Mix: 1:2:4 by dry volume.
Mixing: All batches mixed ½ min. dry and 1 min. wet, for a total of 1½ min. in a No. 0 Smith mixer.
Curing: In molds one day; then placed on strips of wood and subjected to ordinary weather conditions. (September and early October).
Each strength value in table represents average of results of tests on three specimens.

Mix No.	Materials		Grading of Coarse Aggregate	Fineness Modulus of Mix	Slump, in.	W/C	Age	Modulus of Rupture, lb. per sq. in.		Compressive Strength, lb. per sq. in. S	Strength Ratios	
	Sand	Coarse Aggregate						4 x 6 Beam R_1	6 x 8 Beam R_2		R_1/S	R_2/S
19	Janesville	Red Granite	No. 4 to 1½ in.	5.66	3–4	0.87	7 days	372.6	417.3	1544	0.241	0.270
							28 days	522.8	484.0	2652	0.197	0.182
							1 year	681.2	788.4	3873	0.176	0.203
20	Janesville	Lutz Stone	¼ to 1¾ in.	6.03	3–4	0.87	7 days	298.8	341.7	1502	0.199	0.227
							28 days	386.0	486.1	2973	0.130	0.163
							1 year	737.5	732.2	4323	0.170	0.169
21	Limestone Screenings	Lannon Stone	No. 4 to 1½ in.	5.70	3	1.18	7 days	198.7	252.3	834	0.238	0.302
							28 days	291.4	367.2	1384	0.210	0.265
							1 year	467.8	535.3	2062	0.226	0.260
22	Limestone Screenings	Lutz Stone	¼ to 1¾ in.	6.16	3	1.18	7 days	196.7	221.9	859	0.229	0.258
							28 days	340.6	338.2	1545	0.220	0.219
							1 year	486.7	513.0	2090	0.233	0.245
23	Janesville	Trap Rock	No. 4 to 1½ in.	5.77	3–4	0.87	7 days	297.1	297.6	1133	0.262	0.262
							28 days	461.0	475.5	2196	0.210	0.216
							1 year	697.2	685.3	3540	0.197	0.194

points generally fall above the line, and for high strength values they fall below the line. This would indicate that the transverse strength of gravel concrete does not increase in direct ratio to the compressive strength. A similar trend is evident directly in Table 3, for the strength ratios R_1/S are generally lower at the one-year age than at the earlier test ages. If the one-year ratios had been omitted in computing the average, the ratio line would have been raised so that it would fit the plotted points in the lower range more nearly.

The three plotted points representing the mix with Madison sand (shown by triangles) fall in the midst of the alignment, and the points in the strength range of 1,300 to 3,500 lb. per sq. in. in compression indicate good agreement between the transverse and compressive tests.

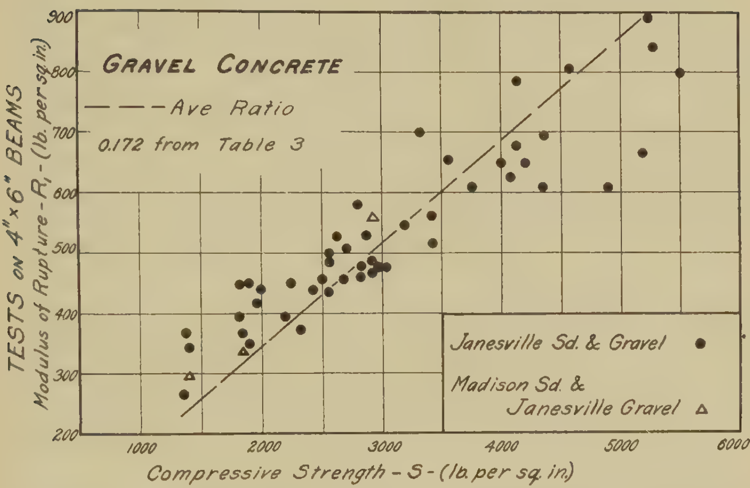

FIG. 2.—RELATION OF MODULUS OF RUPTURE OF 4 X 6-IN. BEAMS TO THE COMPRESSIVE STRENGTH OF CORRESPONDING 6 X 12-IN. CYLINDERS. (GRAVEL CONCRETE.)

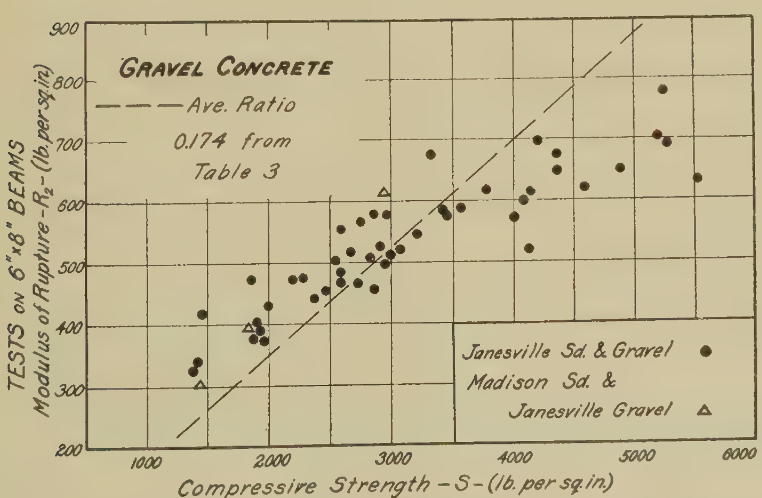

FIG. 3.—RELATION OF MODULUS OF RUPTURE OF 6 X 8-IN. BEAMS TO THE COMPRESSIVE STRENGTH OF CORRESPONDING 6 X 12-IN. CYLINDERS. (GRAVEL CONCRETE.)

In Fig. 3 we have the relation between transverse and compressive strength for 6 x 8-in. beams of gravel concrete. The alignment is nearly the same as for the smaller beams of Fig. 2, there being, however, a more pronounced deviation for high strength values. The average R_2/S ratio, 0.174, is indicated by the dash line across the diagram. It is interesting to note that the 4 x 6-in. beams gave practically the same ratio as obtained from the tests on 6 x 8-in. beams.

By superimposing Fig. 3 on Fig. 2 it would be shown that the two sets of results are in good agreement and both sizes of test beams give fairly consistent values for compressive strengths up to 3,500 lb. per sq. in.

Results similar to those shown in Fig. 2 and 3 appear in a diagram published by the Laboratory of the Portland Cement Association in the Sept., 1927, number of "Concrete Highways and Public Improvements" magazine (inside front cover). A curve is drawn to show the relation of modulus of rupture to compressive strength. This curve agrees with the general trend of plotted points in Figs. 2 and 3, but falls a little below the center of the group.

Transverse strengths of 4 x 6-in. beams of crushed stone concrete are plotted in Fig. 4 opposite the compressive strengths of the cylinders. Most of the points on this diagram lie close to the line representing the average ratio 0.231 from Table 4. A similar alignment is shown in Fig. 5 for the corresponding 6 x 8-in. beams, the average ratio represented by the dash line being 0.229. Comparing the two diagrams it is evident that the two types of beam specimens determined the relation with about equal accuracy.

According to the strength ratios, crushed stone concrete may be expected to develop higher transverse strength than gravel concrete of equal compressive strength. Evidently the bond of the mortar on the rough fractured faces of the crushed stone aggregate results in greater tensile strength, as reflected in higher moduli of rupture, than is obtained with gravel as the coarse aggregate. We may assume that the difference is entirely a matter of bond, because the same sand, and hence the same mortar, occurs in both sets of concretes. The difference in bonding qualities was evident when the fractured beams were examined after test. The crushed stone specimens generally showed tensile failure in a higher percentage of coarse particles than the gravel concrete specimens of the same richness and age. In the gravel concretes some of the pebbles would pull away without fracture, leaving smooth pockets that indicated lack of bond.

Both the 4 x 6 x24-in. and the 6 x 8 x 30-in. beams indicate that the ratio of modulus of rupture to compressive strength is about 0.23 for crushed stone concrete and about 0.17 for gravel concrete. For aggregates whose surface characteristics differ from these materials we would no doubt obtain other ratios.

By placing the diagrams for the crushed stone concretes over the diagrams for the gravel concretes we find that the plotted points fall in the same territory in the low strength range, but they diverge in the high

strength range. This suggests the idea that the modulus of rupture of concrete bears a rather definite ratio to the compressive strength at early ages and for the weaker mixes; in these cases the strength of the mortar evidently governs without dependence upon bond or surface characteristics of the aggregate. When the surrounding mortar is weak, fracture in the aggregate will not occur to any large degree, regardless of the surface characteristics of the pebbles. These considerations may explain why the moduli of rupture of gravel concretes do not increase directly with compressive strength for high strength values, as indicated by the deviations in Figs. 2 and 3.

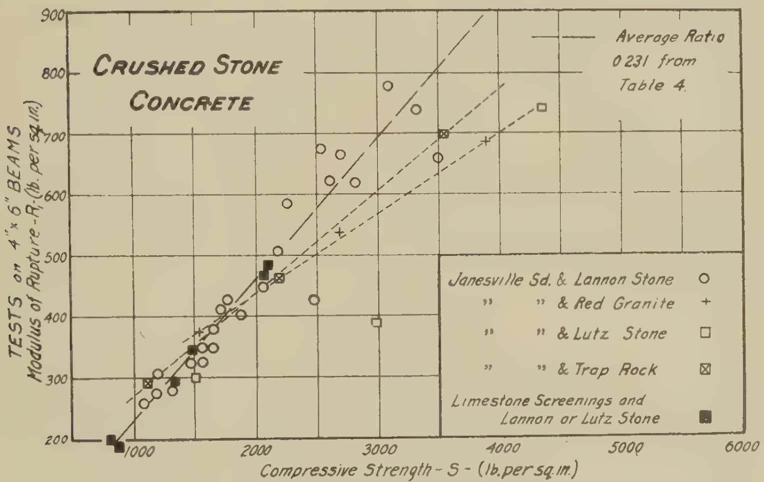

FIG. 4.—RELATION OF MODULUS OF RUPTURE OF 4 X 6-IN. BEAMS TO THE COMPRESSIVE STRENGTH OF CORRESPONDING 6 X 12-IN. CYLINDERS. (CRUSHED STONE CONCRETE.)

Two extra dotted lines are drawn in Fig. 4 connecting the plotted points for mixes made with red granite and with trap rock as the coarse aggregates. Here again the low strength (7-day age) values plot in the group of points near the line representing the average ratio 0.231. But the points for higher strength show a deviation to the right, or to a lower ratio of modulus of rupture to compressive strength. Red granite and trap rock being hard and non-porous, we may say that the difference is due to bonding or surface properties of the coarse aggregate. The same tendencies are shown by the corresponding 6 x 8-in. beams in Fig. 5, but the values are not as consistent as in Fig. 4.

The values obtained on specimens made with Lutz limestone screenings and crushed stone plotted close to the line representing the average ratio, in each case.

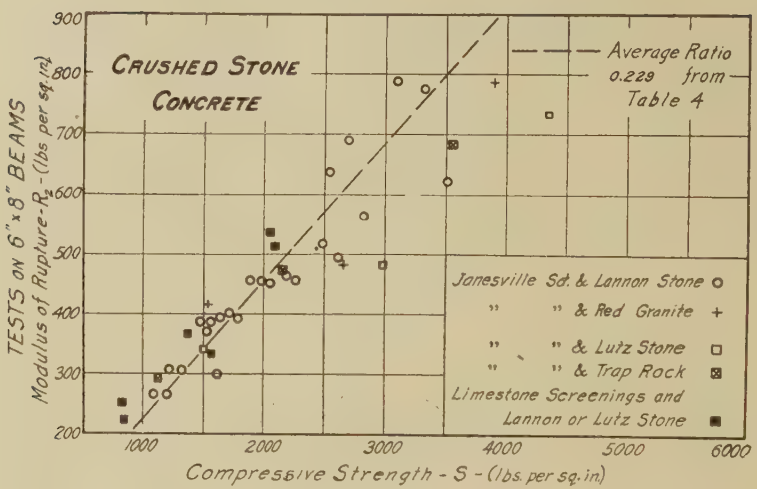

FIG. 5.—RELATION OF MODULUS OF RUPTURE OF 6 X 8-IN. BEAMS TO THE COMPRESSIVE STRENGTH OF CORRESPONDING 6 X 12-IN. CYLINDERS. (CRUSHED STONE CONCRETE.)

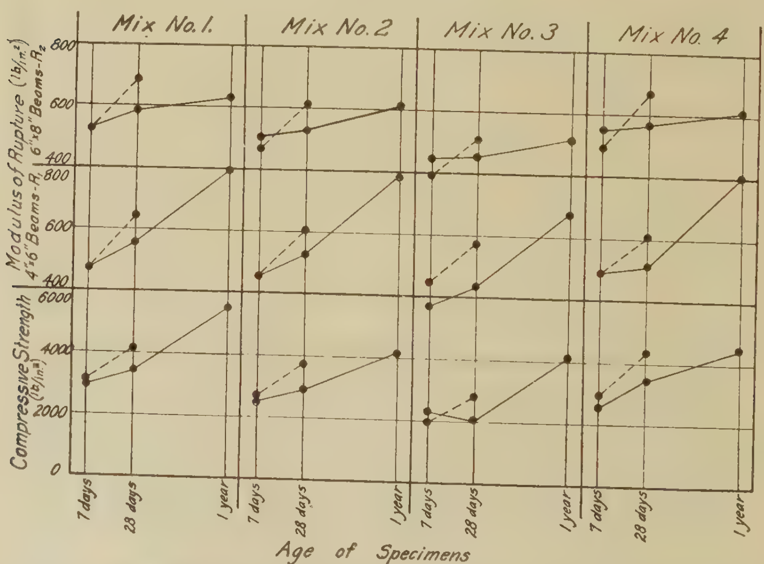

FIG. 6.—INCREASE IN STRENGTH OF CONCRETE WITH AGE, AS MEASURED BY TRANSVERSE AND COMPRESSIVE TESTS. (SOLID LINES FOR DRY CURING, DOTTED LINES FOR DAMP SAND CURING.)

The values of modulus of rupture (R_1 or R_2) and of compressive strength S of the crushed stone concretes of Table 4 are generally less than the corresponding values for gravel concretes of equal water-cement ratio as given in Table 3. These differences are due to the fact that the crushed stone concretes were made in the month of September, in cool weather, whereas the gravel concretes of Table 3 were made in August, in warmer weather. At the one-year age the moduli of rupture for the crushed stone concretes are higher than the corresponding values for gravel concretes.

Strength-Age Curves; Damp Sand Curing.—Figs. 6, 7, 8 and 9 show the average transverse and compressive strengths of all of the mixes at the

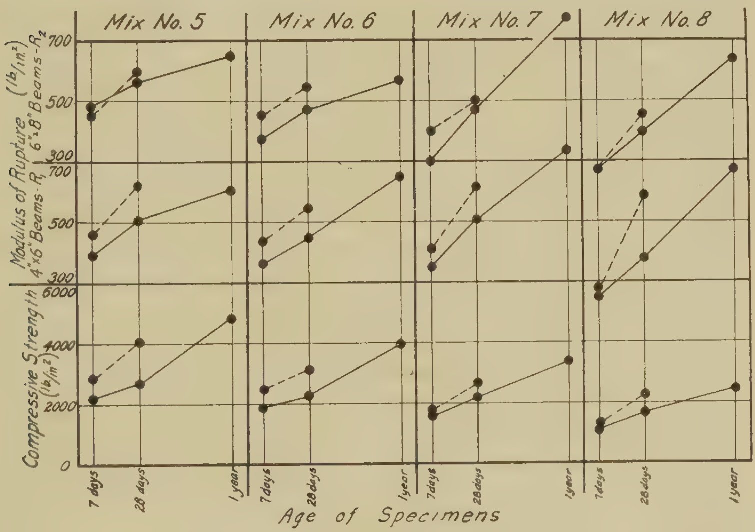

FIG. 7.—INCREASE IN STRENGTH OF CONCRETE WITH AGE, AS MEASURED BY TRANSVERSE AND COMPRESSIVE TESTS. (SOLID LINES FOR DRY CURING, DOTTED LINES FOR DAMP SAND CURING.)

three testing ages. The solid line in each case represents the change in strength for dry-cured specimens and the dotted line indicates the strength for the corresponding specimens cured in damp sand. The upper section for each mix gives the results of tests on the 6 x 8-in. beams, the middle section the results of tests on the 4 x 6-in. beams and the lower section the results of tests on the corresponding 6 x 12-in. cylinders.

We note an increase of strength with age in all of the diagrams. This increase is not the same in all cases, although many of the curves show similar tendencies. The lines representing compressive strength appear to be the more nearly consistent. In Mix No. 3 there is a discrepancy due to a low compressive strength at 28 days. The values of modulus of rupture

from 4 x 6-in. and 6 x 8-in. beam tests show an increase of strength with age for each mix, although the rates of increase are not always alike.

Specimens of mixes 1 to 10 were cured in damp sand for test at 7 and 28 days, with results as indicated by the dotted lines. The strength increases for these specimens were generally much greater than for the corresponding dry-cured specimens, and the rates of increase were usually nearly the same for the different mixes and for the three types of specimens. In many cases the 7-day strength was practically the same for damp sand and for dry curing. Some irregular results are present in the

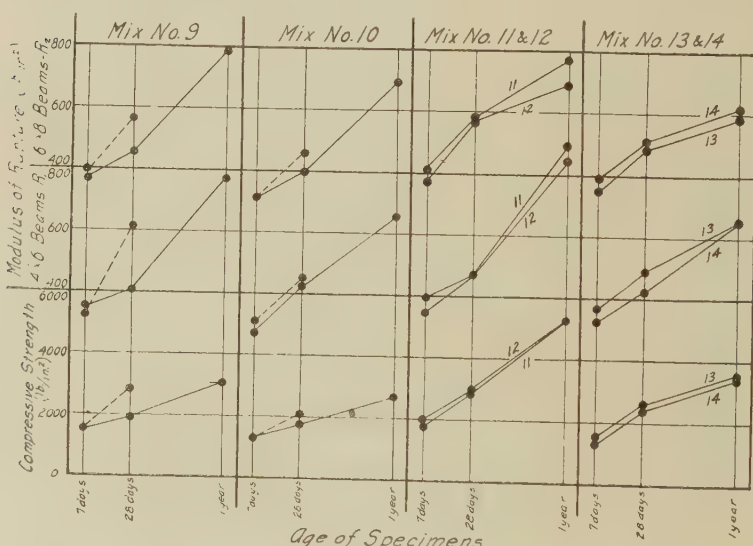

FIG. 8.—INCREASE IN STRENGTH OF CONCRETE WITH AGE, AS MEASURED BY TRANSVERSE AND COMPRESSIVE TESTS. (SOLID LINES FOR DRY CURING, DOTTED LINES FOR DAMP SAND CURING.)

tests, as evidenced by lower 7-day strength for damp sand-cured specimens than for the corresponding dry-cured specimens.

Relation of 7-day to 28-day Strength.—An attempt has been made to relate the 7-day and 28-day strengths of specimens in these tests, in view of the paper presented by W. A. Slater at the 1926 convention of the American Concrete Institute. Fig. 10 shows the relation between strength at the two ages, first for compressive strength, and then for moduli of rupture for the two types of beams.

Mr. Slater suggested the equation:

$$f'_c = f + 30\sqrt{f}$$

for finding the 28-day strength f'_c from the 7-day strength f. He showed

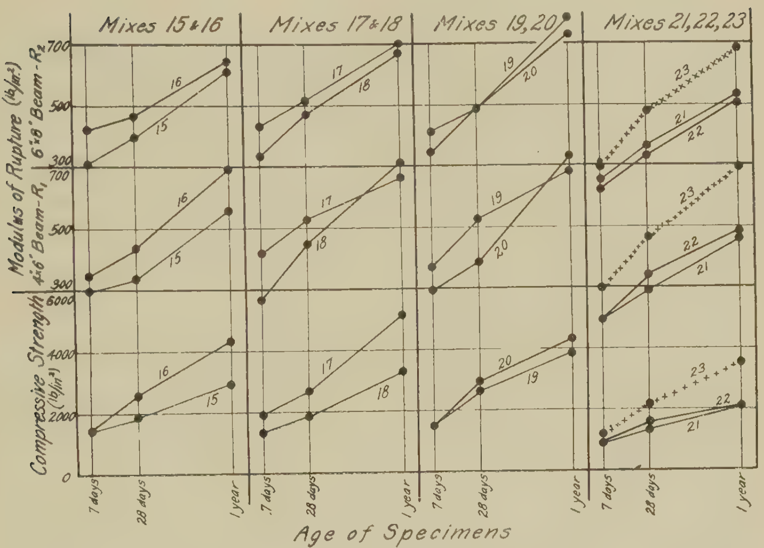

FIG. 9.—INCREASE IN STRENGTH OF CONCRETE WITH AGE, AS MEASURED BY TRANSVERSE AND COMPRESSIVE TESTS.

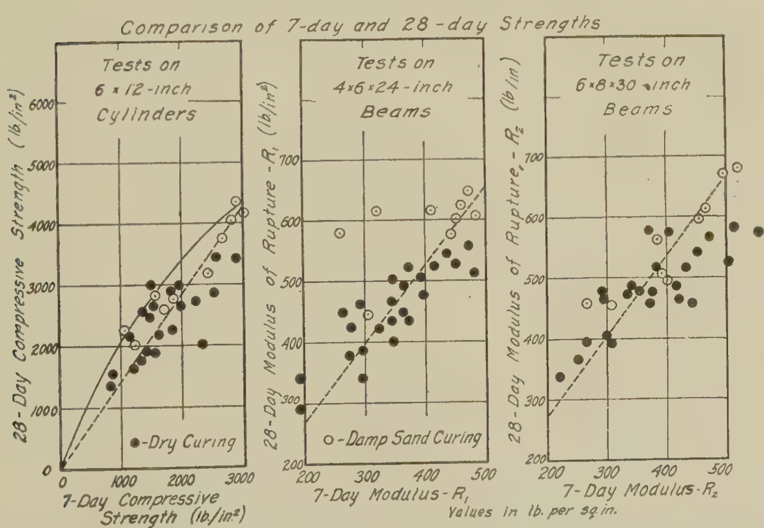

FIG. 10.—RELATION BETWEEN 28-DAY AND 7-DAY STRENGTHS OF CONCRETE, ACCORDING TO TESTS ON 6 X 12-IN. CYLINDERS, 4 X 6-IN. BEAMS, AND 6 X 8-IN. BEAMS.

that the equation gave satisfactory results for concrete cured under normal conditions.

The curve of the above equation is reproduced in the first section of Fig. 10, as the solid line. Some of the points representing specimens cured in damp sand fall near this line, but most of them fall below it. Thus the 28-day strength developed in damp sand curing was usually less than would be expected according to the 7-day results on the basis of the above formula. A few of the points for dry-cured specimens fit the line, but most of them are low, as would be expected. Perhaps the sand curing ought to be conducted by keeping the sand in a saturated condition rather than merely damp, if the quality of the concrete is to be checked by determining its strength-gaining capacity.

The dotted lines on the three parts of Fig. 10 serve to show the trend of strength increase in the concretes of this test, and also to compare the three types of test. Except for two stray points on the plot of 4 x 6 x 24-in. beam tests, the three diagrams show about the same relation between 28-day and 7-day strength. By conducting an adequate series of tests under standard curing conditions it may be possible to show a definite relation of 28-day modulus of rupture to 7-day modulus of rupture. Perhaps such a series would lead to the determination of a constant to replace the factor 30 in the formula $f'_c = f + 30\sqrt{f}$ so that Mr. Slater's formula might be applied to the beam test.

Damp sand curing on the job may be satisfactory for determining the quality of concrete as poured, and it may serve as a substitute for moist closet curing in this respect. However, much of the practical application of the transverse test will be in determining the strength actually developed in the concrete after pouring. For such work the specimens should be cured as nearly like the concrete in the construction as possible. When the concrete is merely allowed to set in the forms and then dry out, without any attempt at curing, the resulting strength relation may be as indicated by the dry-cured specimens of this series, as shown in Fig. 10.

Effect of Grading and Size of Aggregate.—Strength values from the three types of test specimens are shown in Fig. 11 opposite the fineness moduli of the mixes. The data are taken from Table 5, in which the mixes of varying gradation are listed. The water-cement ratio is practically the same for all the mixes in this set.

At the 7 and 28-day test ages the moduli of rupture for the two sizes of test beams show the same strength variation that is indicated by the compression tests. However, the one-year results are not in as good agreement; the discrepancies may be due to the variable conditions encountered in year-round outdoor curing.

Mix No. 11 was made with gravel ranging from No. 4 to ⅜ in. in size; this produced a workable mix which could be easily puddled in the 4 x 6-in. beams, and certainly in the other molds. Mix No. 12 was made with ⅜ to ¾-in. gravel; still a workable mix. Having the same water-cement ratio as Mix 11, this gave practically the same strengths, for each the three

types of specimen. Mix No. 13 had gravel ranging from ¾ to 1½ in. in size; due to the gap in gradation it gave a somewhat lower strength, in all cases, than the other two batches of equal water-cement ratio. Mix No. 14 checked the results of No. 13 closely, the coarse uniform gravel being replaced by coarse uniform crushed limestone, with a mortar of medium strength.

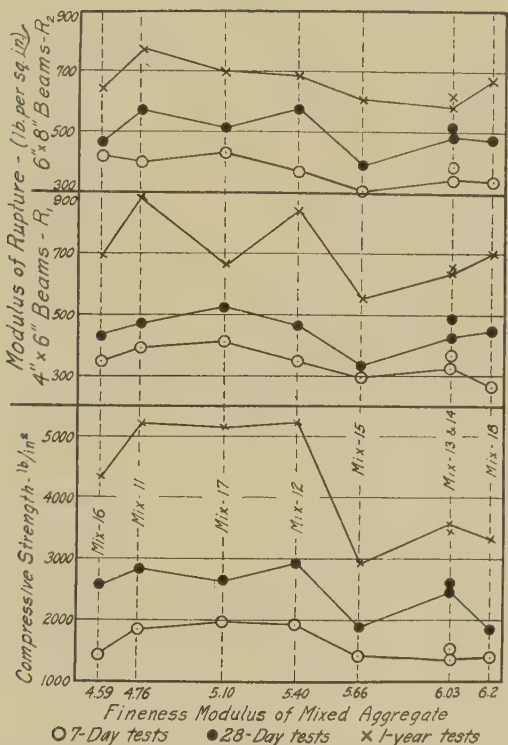

FIG. 11.—BEAM STRENGTH AND COMPRESSIVE STRENGTH FOR MIXES HAVING DIFFERENT VALUES OF FINENESS MODULUS.

The use of Madison sand in Mix No. 15 resulted in low strength, because of the poor quality of mortar due to the excessive fineness of the sand. The two sizes of beams and the compression cylinders showed reduced strength for this mix, in equal measure. On account of the extreme gap in the grading due to the combination of fine sand with uniform coarse gravel this was a mix that easily segregated; still no difficulty was encountered in molding satisfactory test beams and cylinders.

Mix No. 16 was oversanded and the resulting weak mortar produced low strengths in all the specimens. The amount of sand was a little less than the gravel in Mix No. 17; this concrete gave higher strength in most of the tests than other batches of equal water-cement ratio. Mix No. 18 was undersanded, and it gave erratic results because of insufficient mortar to fill the voids in the gravel; this mix was harsh and the specimens were

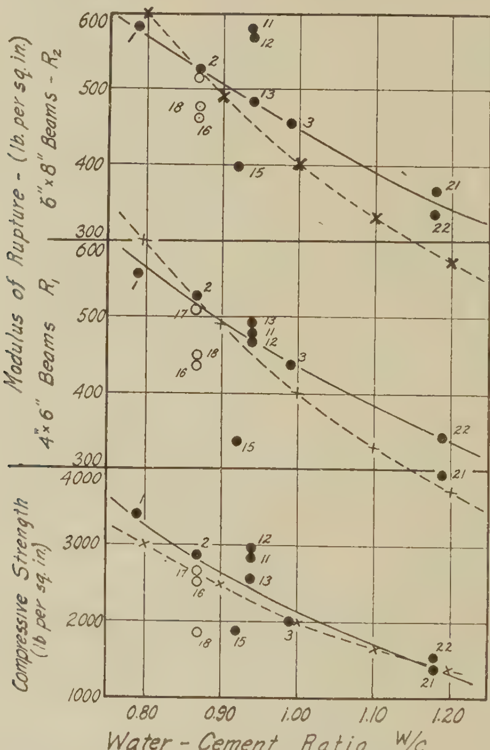

FIG. 12.—THE EFFECT OF WATER-CEMENT RATIO ON THE TRANSVERSE STRENGTH AND ON THE COMPRESSIVE STRENGTH OF CONCRETE.

difficult to make. It is interesting to note that Mix Nos. 11, 12 and 17, for which the fineness modulus was within the "workable range" (about 4.75 to 5.5) gave the highest strengths of all the mixes in this part of the program, as shown in Fig. 11, in all of the tests.

Effect of Water-Cement Ratio.—Twenty-eight-day test results for dry-cured specimens of 1½-min. mix and varying water-cement ratio are shown in Fig. 12. The plotted points are of such alignment that a curve can be drawn showing the relation of the water-cement ratio to the strength of

these specimens. The curves for the beam tests are of the same shape as the curve for the compression cylinders. The plotted points agree with the alignment about equally well in the three cases. Each number next to a plotted point represents the number of the mix. Number 15 plots low in each case, it being the Madison sand mix of poor gradation, and therefore weak in all tests. Mix No. 17 fits the curves; it is the well-sanded mix of Fig. 11. Mixes Nos. 16 and 18, of the same water-cement ratio as No. 17, fall below the curves; they are the over-sanded and under-sanded mixes, respectively.

The original water-cement ratio curve A from "Design and Control of Concrete Mixtures" is shown as the dotted line in the lower part of Fig. 12. It is evident that the strengths in this series of tests are generally higher than the values given by Abrams' curve, with an upward deviation for low values of w/c.

An attempt was made to use a strength ratio for fitting the original water-cement ratio curve to the values of modulus of rupture for the two sizes of beams. However, no agreement between the two lines could be established. The dotted lines in the upper and middle sections of Fig. 12 represent the compressive strengths from curve A multiplied by the arbitrary factor 0.20 as a strength ratio. Inasmuch as the dotted and full-lined curves are not parallel, it appears that the ratio of modulus of rupture to compressive strength is not the same for high and for low values of w/c. The true relation is a problem of mortar strength, bond and aggregate characteristics. If the transverse test is to be used practically in connection with the water-cement ratio method for control of concrete, then it will probably be necessary to make up a modulus of rupture water-cement ratio curve from extensive tests, like the original Abrams' curve for compressive strength.

Some of the results presented in the tables and diagrams in this report are inconsistent, as individual values are frequently in error. When erratic results enter the comparison between transverse and compressive tests, the transverse test values are subject to suspicion. Wide acceptance and long application have established the compressive test as the criterion for practically all of the qualities of concrete. However, it is a question whether the transverse tests do not reveal the fundamental properties of this material in a more definite manner than do the compressive tests.

RECOMMENDATIONS; RESUME.

The 4 x 6 x 24-in. beam can be used effectively for testing concrete of workable consistency having a maximum size of aggregate of 1½ in. A small amount of material up to 2 in. might not interfere with the manufacture of good specimens if the mix is workable. For larger-sized aggregates a correspondingly larger beam should be used, but always with greater depth than width so that the concrete will have enough bulk to puddle easily.

Since practically all concrete construction requires a workable mix, no difficulty will ordinarily be encountered in molding the small-sized beams. Reinforced-concrete columns, beams, floor slabs and piles call for aggregates of a size range that can be puddled properly in 4 x 6-in. specimens.

Some 6 x 12-in. cylinders for compression tests should be made for comparison with beam tests. As soon as the requirements for the beam test have been established, it will be necessary to send cylinders to the testing laboratory only occasionally for check purposes.

If a minimum test load is desired for ease in handling, the 4 x 6-in. beam may be tested flatwise, that is, with 6-in. faces horizontal.

The following is a table of approximate beam loads for various strengths of concrete and sizes of beam, based on an assumed strength ratio of 0.17 (modulus of rupture to compressive strength):

TABLE 7.—APPROXIMATE VALUES OF LOADS IN BEAM TESTS.

	Strength, Pounds per Square Inch		Test load required at end of 6 ft. lever		
W/C	Compression Curve *A*	Modulus of Rupture *R*	4 in. x 6-in. Beam flatwise	4 in. x 6-in. Beam on edge	6 in. x 8-in. Beam flatwise
0.70	3,600	612	136	204	408
0.80	3,000	510	113	170	340
0.90	2,500	425	95	142	284
1.00	2,000	340	75	113	226
1.10	1,700	289	64	96	192

The ratios of modulus of rupture to compressive strength for gravel concrete and for broken stone concrete may be assumed to be 0.17 and 0.23 respectively, unless tests have been made to determine the ratios directly for the particular aggregates under consideration. If specimens are to be cured in damp sand for determining the quality of concrete, then the sand pile must be saturated in order to give conditions which will lead to strength values such as obtained in moist closet curing.

When it is desired to determine the strength actually developed in the structure, it is imperative that the test specimens be cured adjacent to the poured concrete which they represent, and conditions surrounding the test specimens must be as nearly as possible like the conditions governing the rate of strength increase in the structure. Differences due to exposure of the test pieces and due to massiveness or thickness of the structure should be kept in mind. The concrete in the structure generally has a better chance to develop strength than the concrete in the specimen, and therefore errors due to test conditions will be on the safe side.

The practical value of field testing depends largely upon the promptness with which the results are made available. Any improvement in practice due to more convenient and more simple equipment and due to stand-

ardized procedure will lead to corresponding benefits to the manufacturer of the concrete and to the ultimate owner of the construction.

Transverse test values are a direct measure of the strength of the structure in the case of road slabs. For constructions in which tensile or transverse loadings are of secondary importance, the values of modulus of rupture from beam tests can be used as indirect measures of quality, by means of the strength ratio R/S.

ACKNOWLEDGMENTS.

These tests were made in the laboratory for testing materials at the University of Wisconsin, as a part of the research work of the department of mechanics. The writer is indebted to Prof. M. O. Withey for many valuable suggestions. Messrs. L. J. Heise, R. E. Richardson, Moulton Basford, G. S. Paul and R. E. Greiling assisted in the manufacture and testing of the specimens and in many of the computations. Donations of aggregate from George Nelson, Madison, Wis.; Trap Rock Co., Minneapolis, Minn.; Lutz Stone Company, Oshkosh, Wis.; and Wisconsin Granite Company, Chicago, Ill., for portions of the program, are gratefully acknowledged.

Mr. Gilkey. HERBERT J. GILKEY.—This paper contains much of interest and value. The writer desires to comment briefly upon the portion of the paper that pertains to age and method of curing specimens. As regards the general content of the paper, this is but a minor and subordinate feature.

In practically all, if not all cases, the one-year tests of air-cured specimens show strengths considerably above the 28-day strengths, either moist cured or air cured. This showing is liable to give what the writer believes to be a wrong impression, to some at least. As the author states in the paper, the one-year air curing did not mean exposure to dry air for the full year. Atmospheric moisture (fog, rain, snow, etc.) doubtless furnished considerable moist curing during the year. Moreover, the specimens were probably quite air dry at time of test and would therefore show up stronger than the same specimens tested moist. There is much to be said in favor of testing specimens in the same condition as the concrete that they represent, but it would also be very interesting to know (1) What strengths companion specimens to these air-cured one-year specimens would have given after a few hours or a day in water just prior to test; (2) What strengths companion specimens moist cured for the full year and tested wet would have shown.

The fact that the natural moisture from the elements and from the subgrade (the latter not present in the case of these specimens) aids greatly in the ultimate curing of pavements, sidewalks and other exposed construction, has doubtless been a real factor in producing good structures in spite of frequent neglect as to the initial curing period and method. The same compensating factors are not present in the case of most inside construction. Any strength not acquired in the short period of initial curing in an indoor structure is likely to be permanently missing. Moreover, a pavement is liable to get its most severe test at a fairly early age and when it and the subgrade are in a saturated condition.

The closeness of strengths of 7-day specimens moist cured and air cured was mentioned. This is a very natural thing. The 7-day air-cured specimen has probably not dried out fully enough to entirely halt normal strength gain, so it differs little from the moist-cured 7-day specimen in status. Insofar as it has dried out the strength has been increased by mechanical hardening due to drying, and this probably offsets any loss in strength due to interrupted curing. An accurate measure of the efficiency of curing methods must of necessity be obtained from fairly long-time curing tests after strength gain due to normal curing has become quite gradual. Otherwise, the added curing of the moist specimen tends to be offset by the added strength due to drying of the other. The strength from the added moist curing is more or less a permanent asset, while that due to drying is temporary since a short period of soaking will bring the specimen back

to the saturated strength. There have been a number of instances in which this unrecognized overlapping of opposing factors has been a source of misleading interpretation of test data.

There are serious gaps in our knowledge of curing. From very limited data and random sources there is considerable evidence to the effect that:

(1) At some humidity concrete exposed to air will neither increase nor decrease in strength after once it has dried to approximately constant weight for that humidity.

(2) At humidities above this the concrete will slowly absorb moisture from the air and there will be a gradual strength increase due to continuing hydration.

(3) At lower humidities (drier air) there is a gradual falling off of strength, the reasons for which are not at present known but one of which is possibly a gradual deterioration of crystal structure.

(4) Oven drying at even relatively low temperatures causes appreciable falling off of strength, and it is therefore not feasible to attain the added strength of dry concrete over wet through a process of oven drying. The explanation for this when found will probably be closely allied to that for (3).

It is the writer's feeling that these four tentative facts define a need for an investigation designed to: (1) Check them; ascertain whether or not they are correct statements of fact; (2) Define their measure, i. e., evaluate the extent of the effects and find the range over which the specimen may remain at constant strength; (3) Ascertain whether all cements and mixtures are subject to similar changes under similar conditions; (4) Co-ordinate the flexural and the compressive tests with regard to these several factors. The present paper furnishes some excellent elementary data for a preliminary comparison.

As will be readily recognized, such work as has been suggested could only be undertaken in an unusually well-equipped laboratory. Accurate humidity control requires much in the way of specialized equipment. Prof. Raymond E. Davis, of the University of California, has been doing some work much of which will doubtless throw light upon some of the questions raised.

It is apparent that the present discussion has been somewhat apart from the intent, scope and data of Mr. Wiepking's excellent paper. Nevertheless the data of the paper furnish evidence of the need for more knowledge along the lines discussed. No portion of the discussion is intended as a criticism, by inference or otherwise, of any part of this paper. Parts of the discussion may assist in the interpretation of some of the data. In a variety of places there is available some literature upon the topics mentioned.[1]

[1] *Proceedings*, A. C. I., 1926, pp. 395-436; *Proceedings*, Am. Soc. C. E., Jan., 1927, pp. 79-93; *Transactions*, Am. Soc. C. E., Vol. 91 (1927), pp. 153-167; *Engineering News-Record*, Dec. 1, 1927, pp. 879-880 (P. J. Freeman); *Proceedings*, A. S. T. M., 1927, Vol. II, p. 331 (P. H. Bates); *Proceedings*, A. S. T. M., 1927, Vol. II, pp. 424-428; Bulletin No. 43, Univ. of Washington Eng. Experiment Station (A. L. Miller and H. F. Faulkner).

The Carrying Capacity of Semicircular Hooks.

By T. D. Mylrea.*

1. *Introduction.*—From almost the beginning of reinforced-concrete construction, anchorages for the ends of the bars have been used. The tie-rods and vertical hangers of bowstring arches, the reinforcing bars in the corners of rigid frames and bents, and the bars in beams of variable depth all furnish examples where the use of anchors is obviously necessary. It is in these cases that anchors are put to their most severe use, for it is here that they must develop the entire stress in the bars and do so without slip.

In the case of ordinary beams it was early realized that the member was stronger if the bars were anchored, and the idea of increasing the safety of a structure by the use of anchors is found in the earliest works on the subject. Thaddeus Hyatt, for example, in 1876 and 1877 tested beams in which anchors were employed. He got his best results from beams in which flat main reinforcing bars were bent up at right angles at the ends, each end of each bar being provided with a small knob similar to a bolt head. Hyatt's designs were based on a vague notion as to the interaction of steel and concrete; yet to this day the design of anchorages has remained largely a matter of guesswork. The problem is not confined to the pull-out strength of an anchor, but is intimately associated with bond strength, bar strength, and slip of bar.

While the factors of tension, compression, and shear have been reduced to a fairly satisfactory approximate basis for purposes of design, bond and anchorage are factors which will bear much study. Important as it is, many designs may still be found in which bond seems to have been wholly neglected. It was suspected by W. F. Scott as early as 1907 [1] that the bond in beams is not distributed in the manner indicated by the usual bond formula, and the experiments of Bach [2] and Abrams [3] fully demonstrate that such is the case. It was found that unanchored bars slipped, that when anchors were used these anchors were subject to stress, and that such anchors tended not only to prevent bond failure, but secondary failures resulting from slip. It seems only reasonable, for example, when to the natural elongation of the tension side of a beam is added an increment due to end slip that tension cracks will penetrate much further into what would normally be the compression area of the beam.

The question of anchorage in beams naturally falls into two parts; first, how much stress is carried by the anchor, and, second, what con-

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* Professor of Building Construction, Carnegie Institute of Technology.
[1] *Transactions*, A. S. C. E., Vol. LXXIII, 1910.
[2] Deutcher Ausschus fur Eisenbeton, Hefts 9 and 10, 1911.
[3] University of Illinois, Bulletin 71, 1913.

stitutes an effective anchor? The first part of the question has been given various answers, in each successive case the answer being larger. It is not the purpose of this paper to enter into a discussion of this phase of the subject beyond making the following two statements. With steel of high elastic limit it is possible for the stress at the anchor to be almost as high as at midspan before passing the elastic limit, and with the use of higher working stresses more and more dependence must be placed upon anchorage. Secondly, it is not much more difficult to devise an anchor capable of carrying a high stress than one carrying a low stress. The writer is in favor of anchors capable of carrying the higher stresses.

The question of what constitutes an effective anchor is one about which there has long been a diversity of opinion, and many forms have been devised. In this country, possibly as a result of Hyatt's tests and the ease of fabrication, the simple right angle bend has had the greatest favor. European thought on the subject is very well represented by the following quotation from E. P. Goodrich's translation of "Der Eisenbetonbau" by Professor E. Morsch, 1908.

"The ends of reinforcing rods should always be made with a hook so that sole dependence is not placed on friction or adhesion. For this purpose the shape of the hook is of importance. The form commonly employed, of a simple right-angle bend, is not very effective when surrounded only by a thin concrete slab, as is often the case at the ends of beams. In such cases the ends should rather be given a larger bend of as much as 90 deg. Considére, in the French section of the International Society for Testing Materials, reported a new form of the end hook, which should be immediately adopted in practice. By bending the end into a half circle, to which a short straight piece may be added, the principle of rope friction is employed and a greater frictional resistance is produced on the inner side of the bend, since the hook will be pressed hard against the concrete. . . . These hooks possess the further merit of not depending to any great extent upon the character of the concrete or the care given the work, since a rope-like friction is secured by the large curve of pressure. This pressure naturally should not be too large, since then a crushing of the concrete results. According to Considére, the best results are secured by giving the semi-circular bend a (clear) diameter about five times that of the rod."

The report of the First Joint Committee on Concrete and Reinforced Concrete, presented in 1916, contains the following statements: "Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorages may properly be used in special cases. Anchorage furnished by short bends at right angles is less effective than by hooks consisting of turns through 180 deg."

According to the report of the New Joint Committee a "special anchorage" is considered satisfactory if it provides for a stress equal to one-third the maximum working stress by means of an added length of embedment, either straight or bent, on condition that the radius of bend be not

less than four bar diameters. This provision of a minimum radius of bend is quite in line with the opinion of Morsch. Quoting again, "As is shown by the beams . . . of the author's experiments, it is necessary to keep at a low value the compression of the concrete at the bend of the rod. With $f_s = 14{,}223$ lb. per sq. in. (translated from metric figures) and p

TABLE I.—RESULTS OF THE RESEARCH UPON BOND STRENGTH AND HOOK RESISTANCE (SALIGER).

Specimen	Description	Diameter of Hook	Tensile Force, in metric tons: First Crack	Failure, Each	Failure, Average	*Maximum Bar Stress, in terms of $f'c$	Stresses at Failure kg/gcm: fs	u	Ratio of Loads Carried		Appearance at Failure
a 1	St.	......	3.60	3.60	4.14	2.52	450	18.0	1	.....	Vertical crack.
a 2			4.68	4.68		3.26	583	23.4			Iron pulled out.
b 1	H.	4.37d	5.50	5.84	6.07	4.08	730	....	1.46	1	Crushed at the middle.
b 2			5.42	6.30		4.38	785	....			
c 1	H.	4.37d	6.49	10.65	9.67	7.43	1330	....	2.33	1.59	Crushed at the middle.
c 2			6.20	8.70		6.04	1080	....			
d 1	Sp.	4.37d	10.20	†	12.80	.....	....	....	3.08	2.11	General bursting.
d 2			11.00	12.80		8.88	1590	....			
e 1	Sp.	4.37d	13.50	14.60	14.60	10.16	1820	....	3.52	2.41	General bursting.
e 2			8.50	†		.....	....	....			†Load not observed.
f 1	St.	....	....	3.10	3.01	3.26	584	18.9	1	....	Iron pulled out, f_1 without crack, f_2 with vertical crack.
f 2			2.92	2.92		3.07	550	17.8			
g 1	H.	3.95d	3.97	4.62	4.09	4.86	870	....	1.36	1	Crushed at the middle.
g 2			3.30	3.55		3.74	670	....			
h 1	H.	3.95d	2.50	5.00	5.71	5.25	940	....	1.90	1.39	Burst.
h 2			5.41	6.41		6.77	1210	....			
i 1	Sp.	3.95d	9.30	9.70	9.85	10.16	1820	....	3.27	2.41	Burst.
i 2			6.32	9.99		10.50	1880	....			
k 1	St.	....	....	3.95	3.61	6.98	1250	31.4	1	....	Iron pulled out with no cracking.
k 2			....	3.27		6.81	1040	26.0			
l 1	H.	5.0d	4.92	4.92	4.90	8.81	1560	....	1.36	1	Crushed at the middle.
l 2			4.88	4.88		8.66	1550	....			
m 1	H.	5.0d	5.65	5.85	5.95	10.40	1860	....	1.65	1.21	Burst.
m 2			5.71	5.95		10.55	1890	....			
n 1	Sp.	5.0d	9.34	9.34	9.65	16.52	2960	....	2.67	1.97	Burst.
n 2			9.20	9.95		17.65	3160	....			
o 1	St.	....	....	4.49	4.54	12.50	2240	44.9	1	....	Iron pulled out with no cracking.
o 2			....	4.59		12.72	2280	45.9			
p 1	H.	6.25d	5.15	5.15	5.18	14.35	2570	....	1.14	1	Crushed at the middle.
p 2			5.20	5.20		14.40	2580	....			
q 1	H.	6.25d	4.82	4.82	5.16	13.46	2410	....	1.13	1.00	Burst.
q 2			4.95	5.50		15.30	2740	....			
r 1	Sp.	6.25d	6.23	7.98	8.14	22.20	3970	....	1.79	1.57	Burst.
r 2			5.60	8.29		23.00	4120	....			

The specimens were made of mortar and tested at 54 days' age. The ultimate compressive strength ($f'c$) of 20 cm. cubes was $\frac{1}{6}(215 + 212 + 161 + 170 + 155 + 163) = 179$ *kg/gcm.*

* This column was added by the writer.

(the pressure under the bend) equal to 853 lb. per sq. in., $r = 13d$, approximately." Providing for one-third of a working stress of 18,000 lb. per sq. in. with an allowable pressure of 900 lb. per sq. in. would require a radius of:

$$\frac{1}{3} \times \frac{18{,}000}{14{,}223} \times \frac{853}{900} \times 13d = 4d.$$

It is, of course, evident that if at any time the stress at the junction of the hook with the straight portion of the bar were to exceed 6,000 lb. per sq. in., then a radius of more than $4d$ would be required. This too is predicated on the assumption that a pressure of 900 lb. per sq. in. is permissible under the bends, 900 being the value permitted in negative compression by the New Joint Committee report.

2. *Brief Résumé of Previous Tests.*—Several noteworthy programs of experiment with anchored bars have been reported, some dealing with pull-out tests only, and others dealing with beams. Of the first type probably the most extensive series of tests of semi-circular hooks is that reported in 1913 by R. Saliger in "Schubwiderstand und Verbund in Eisenbetonbalken." Of the tests to determine the effects of hooks in beams, those of

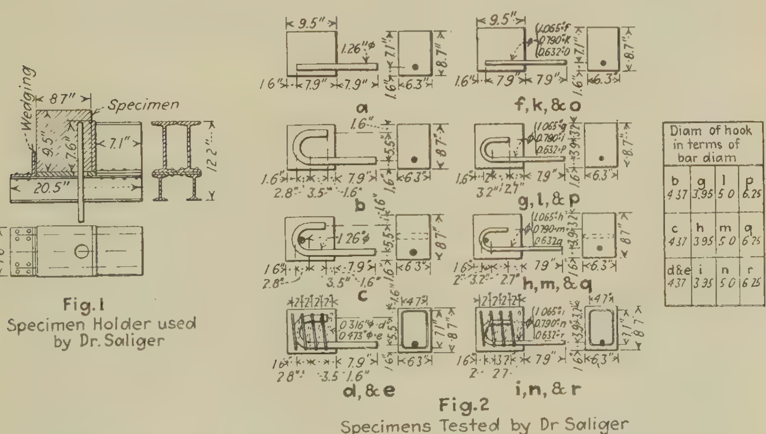


FIG. 1. FIG. 2.

SALIGER SPECIMEN HOLDER AND TEST SPECIMENS.

C. Bach, reported in "Hefts" 9 and 10 of the "Deutscher Ausschuss fur Eisenbeton," 1911, are perhaps the most noteworthy, while those of Faber, reported in "Reinforced-Concrete Beams in Bending and Shear," 1924, also have much of interest. The tests of Abrams, University of Illinois Bulletin No. 71, 1913, while referring primarily to bond, have also an important bearing upon the question of anchorage.

In Figs. 1 and 2 are shown the form of specimens and holder used by Saliger. The record of his tests is given in Table I. The dimensions in Fig. 1 are the English equivalents of the metric units used by him, while in Table I the stress unit of kilograms per square centimeter has been retained. This may readily be converted into pounds per square inch by multiplying by 14.22. The fact that atmospheric pressure is so nearly 1 kg. per sq. cm. has at times lead German authors to speak of the magnitude of stresses as being of so many "atmospheres."

In Saliger's tests the specimen in each case was very little larger than the hook, thus eliminating the effect of a long straight embedment below the hook. It is possible, however, that in this manner other effects were introduced. A compression bearing so close to the anchorage might be expected to exert a restraining effect similar to that noted in the compression tests of cubes. The small size of the specimens was conducive to cracking, particularly in those cases where the size of the bar was proportionately large, and it will be noted that all hooked specimens failed by splitting. Table I shows, nevertheless: (1) that the bars having the largest radius of bend in terms of bar diameter carried the greatest unit stress, at least up to hooks with $r = 6.25d$; (2) that the wire binding added to the carrying capacity; and (3) that in those specimens where the binding varied, as in d and e, the specimen with most binding carried the greatest load. No slip measurements were taken.

The Bach and Graf beam tests showed increases of 50 per cent or more in carrying capacity when the ends of the bars were anchored by means of hooks, the form of the hook not having a great deal of effect. No spirals were used. This checks fairly well with Saliger's data on unspiralled pull-out specimens, and leads at once to the question: "If Saliger's spiralled specimens carried so much more than his unspiralled specimens, would not the same effect have been obtained in Bach's beam tests?" It may be strongly suspected that such would have been the case. In other words, the unspiralled hook, of no matter what form, does not function at its best as an anchor, and it is quite probable that beam strengths may be increased beyond anything now known by the use of proper and thoroughly efficient anchors. This is confirmed by the fact that every one of Bach's beams was split by the hooks. Fig. 3 [4] well illustrates this action.

3. *Purpose of the Program, and Acknowledgments.*—Now, to be absolutely dependable an anchor must (1) be capable of developing in the bar a stress at least as high as the elastic limit (preferably the ultimate strength), (2) it must do so without damaging the concrete, and (3) it must do so with the absolute minimum of slip in order to preclude secondary failure. Semi-circular hooks are known to be able to carry a certain amount of load, which amount is rather indefinite, but is supposed to bear some relation to the radius of bend. They are known to split the concrete when the mass to be split is small; but comparatively little is known as to the amount of slip accompanying stress in the hook. In a previous paper [5] the writer attempted an analysis of the splitting effects of hooks, based on three different assumptions. These assumptions were admittedly "feelers," and the analysis was but a study. However, it lead to the conclusions that the splitting effect of a hook might be quite large in proportion to the bar stresses, and that to reinforce against bursting, when the bar stress at the hook was 40,000 lb. per sq. in., in a spiral cage having Na_s equal to twice the area of the bar would be required, N being

[4] From "Deutscher Ausschus fur Eisenbeton, Heft," by Bach and Graf.
[5] *Journal*, Western Society of Engineers, Jan., 1926.

the number of turns in the spiral, and a_s being the cross-sectional area of the spiral strand. It was pointed out, too, that with such a bar stress the pressure under the hook would be very high. The problem is too complex to admit of a complete solution by the theory of elasticity, and

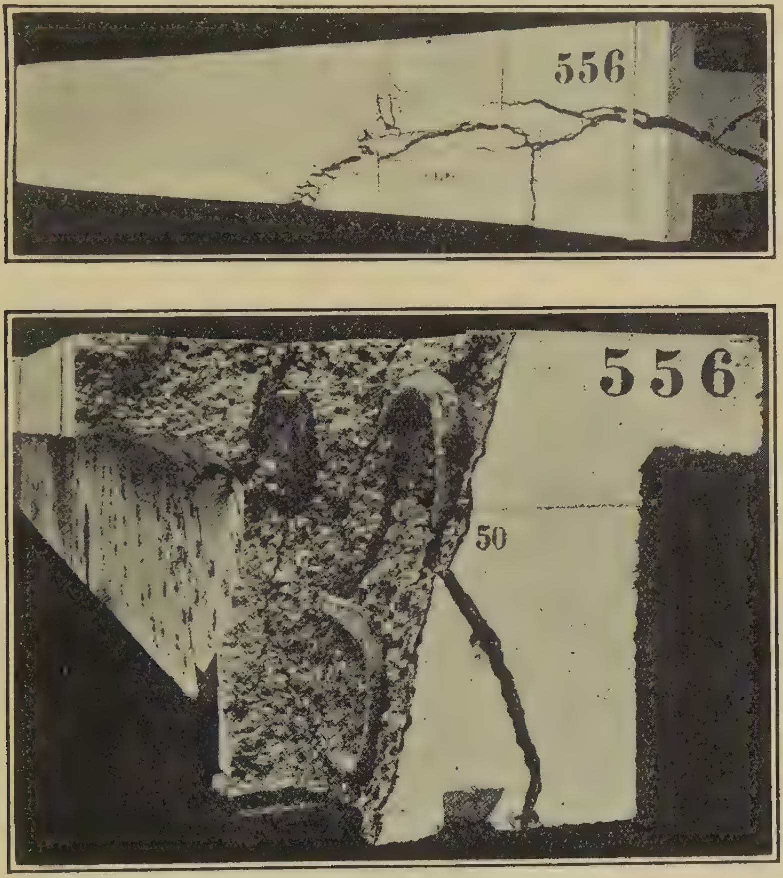


FIG. 3.—T-BEAM SPLIT BY HOOKS.

it is necessary to resort to approximate analysis backed by experimental investigation.

For these reasons it was considered desirable to conduct a series of tests upon semi-circular hooks of various sizes, reinforced with spiralling of varying amount. The work was undertaken by C. J. Posey in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering in the graduate school of the University of Illinois dur-

ing the school year of 1926-27, under the direction of the writer. Mr. Posey is to be commended for the careful work which contributed largely to the success of the program. Acknowledgment is due also to Dean Milo S. Ketchum, of the college of engineering and Director of the Engineering Experiment Station, and to Prof. W. C. Huntington, head of the department of civil engineering, for permission to publish the results of the tests.

4. *The Specimens.*—The specimens used in the present investigation were all of one standard size, and certain features of design were common to all. These standard features are shown in Fig. 4. In order that the specimens might not be unwieldy the bar size was fixed ½ in., which proved quite satisfactory. High carbon steel, having an elastic limit of

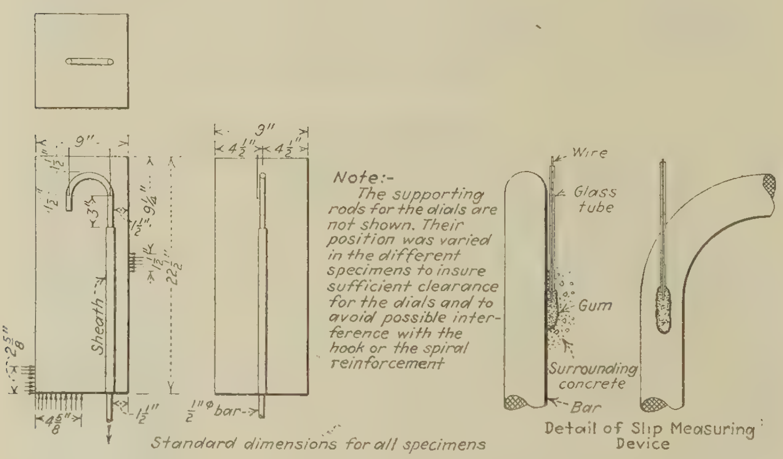


FIG. 4.—STANDARD FEATURES OF DESIGN.

70,600 lb. per sq. in. and an ultimate strength of 112,000 lb. per sq. in. was used in every case in order to obtain experimental data over as large a range of value as possible for each anchorage. Early tension failure would defeat this end. In order to minimize any possible effect of proximity of the compression face to the hook, the length of the specimen was made such that the center of the largest hook (12*d*) was three times the diameter of the hook from the end bearing on the support.

Since the investigation was confined to the investigation of the action of the hook proper it was necessary to eliminate the effect of bond on the long straight embedment below the hook, so this portion of the bar was enclosed in a sheath consisting of three thicknesses of blotting paper. The sheath in all cases extended up the bar to a point three inches below the point of tangency. In the case of the flat bend it was necessary to have some straight embedment below the end of the hook, while in the case of the larger hooks it was felt that this three inches would take care

of the lateral pressure on the straight bar below the point of tangency caused by the slight pulling around of the hook. This pressure is an effect of the anchorage and must be considered a part of it. Just how far down the side of the bar it is significant is a matter of conjecture, but in a distance as great as six times the diameter of the bar it should practically disappear.

It was found that the friction of the bar in the sheath was not great. In one of the friction tests a pull of 80 lb. was the maximum required to kept the bar sliding. In the other, a pull of 90 lb. caused a slip of 0.35 in., and 200 lb. was the running maximum. The friction of the sheath was therefore neglected.

As one of the problems to be investigated was the effect of variations of diameter of the hook, it became necessary to decide whether or not the size of the specimen should be varied with the changing diameter of hooks. For the sake of simplicity and uniformity in casting and testing the specimens, it was decided to adopt a uniform size. The specimen was made sufficiently large to accommodate a hook of diameter twelve times that of the bar to be used. (See Figs. 6 and 7.) Hence the size of the test specimens became nine inches, or eighteen times the diameter of the bar. The specimens were made square.

A feature calling for careful consideration in the design of the specimen was the method of measuring the slip. In the beam tests by Bach and Graf, small conical wooden plugs were cast in place, which when withdrawn would expose the steel where slip measurements were desired. This device was considered unsatisfactory for the purpose of the present investigation, since the presence of a large hole left by the withdrawal of large wooden plugs is conducive to the formation of cracks. In these tests it was considered desirable to measure the slip at the beginning of the bend, in which case the wooden plugs would be long and consequently quite large.

The expedient that was employed is illustrated in Figs. 4 and 5. A wire was attached to the rod. It was bent sharply at right angles about ⅛ in. from the surface of the rod, and was carried off in a glass tube parallel to the axis of the rod. The glass tube protected the wire from contact with the concrete except at the bend. At this point the wire was protected and the bottom of the glass tube sealed by a plastic adhesive gum. The volume of the gum was sufficient to permit 0.4 or 0.5 in. travel of the wire in the direction in which the bar was expected to move. The gum that adhered to the side of the rod covered about 0.1 sq. in. of bar surface in most cases, and its effect was probably negligible. The tendency of the glass tubes to weaken the concrete might be suspected. However, though these glass tubes were in the general plane of splitting in every case, in no case was the crack found to have followed a glass tube for even a fraction of its length. The glass tubes varied in diameter, but the largest ones used were not over ⅛ in. outside diameter.

In order to provide steady supports for the measuring dials, small steel rods were embedded 1 in. or so in the top of the specimen at some

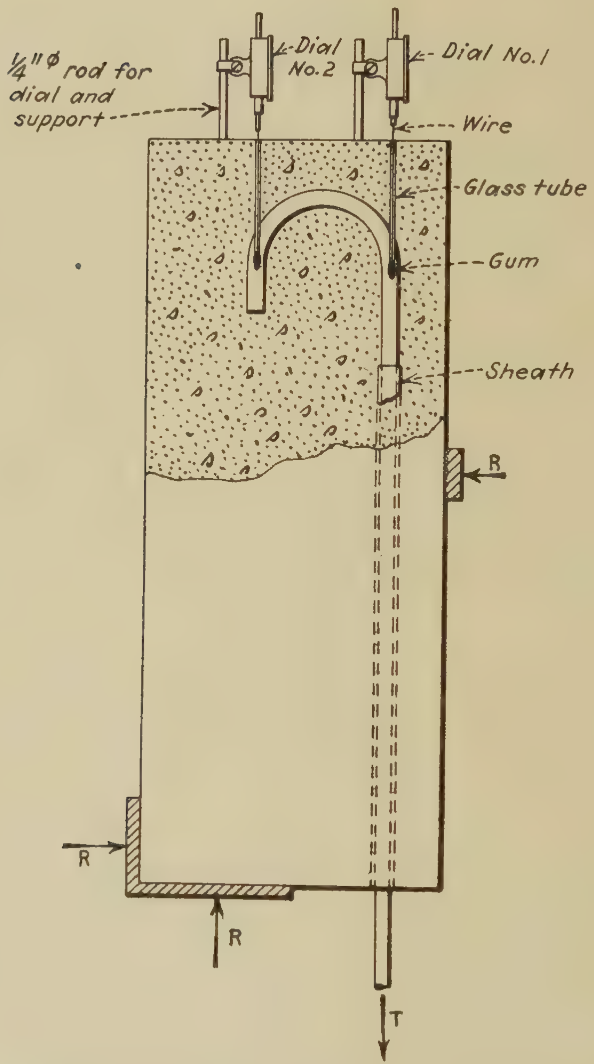


FIG. 5.—DEVICE FOR MEASURING SLIP OF HOOK.

distance from the points of emergence of the wires and tubes (Fig. 5). The position of these rods was not fixed; but they were always placed to one side and away from any possible interference with the interior reinforcement. They did not seem to have any effect on the formation of cracks. Fig. 11 shows the rods with the dials attached.

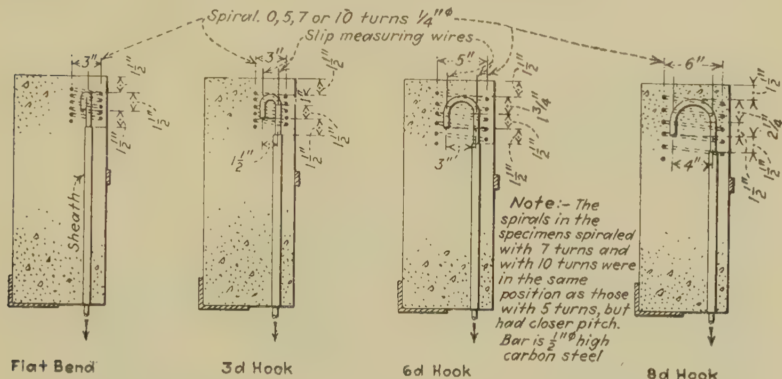


FIG. 6.—DETAILS OF SPIRALS FOR DIFFERENT HOOKS.

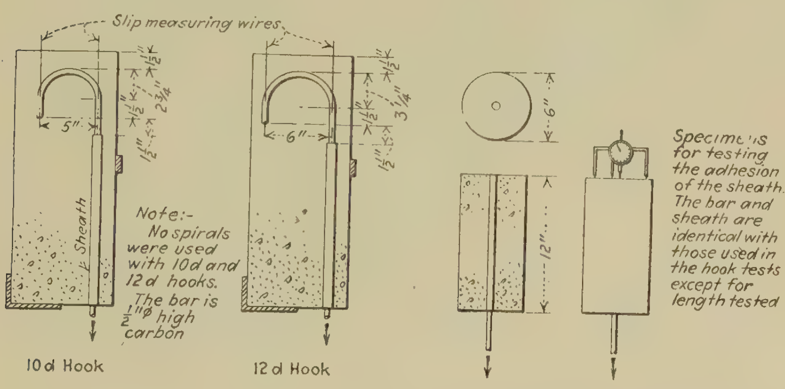


FIG. 7.—DETAILS OF SPIRALS, AND, AT EXTREME RIGHT, PULL-OUT SPECIMEN TO TEST ADHESION OF SHEATH.

The specimen holder, shown in Figs. 8 and 10 was designed with the idea of approximating stress conditions at the end of a beam. Only one-half of the base of the specimen is in bearing; this approximates the compression distribution in a beam and precludes the wedging action which often occurs in pull-out tests. The bearing block on the side is below the position of the hooks, leaving the top portion of the specimen free for in-

spection. In this respect the holder was different from Saliger's, the backs of his specimens being tight in the holder for their full height.

The sizes of hooks tested were: Flat bend, 3*d*, 6*d* (Considère), 8*d* (Joint Committee), 10*d*, and 12*d*. The flat bend, 3*d*, 6*d*, and 8*d* hooks were also tested when spiraled with 5, 7, and 10 turns of ¼-in. low-carbon steel wire, the 10-turn spiral having Na_s a little more than twice the area of the bar. In order that the spirals might be as effective as

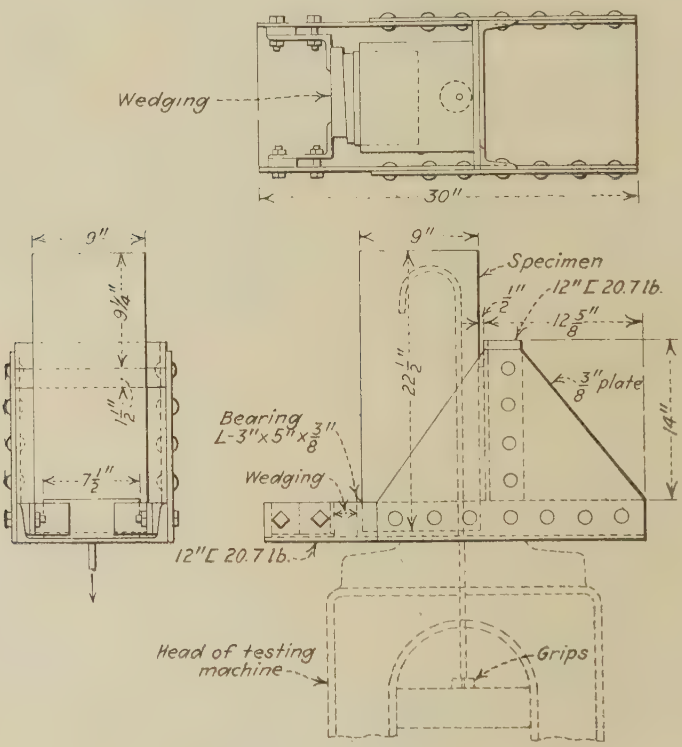


FIG. 8.—ARRANGEMENT FOR HOLDING TEST SPECIMENS.

possible, they were made just large enough to clear the hook, except in the case of the flat bend. Detail drawings of the spirals for the different hooks are given in Figs. 6 and 7. A small pull-out specimen was designed to test the friction of the blotting paper sheath. This specimen is illustrated in Fig. 7.

5. *Preparation.*—Four molds being available, the specimens were cast and tested in groups of four in the following order:

(1) Series I without spirals: 3*d*, 6*d*, 8*d*, and 12*d* hooks.

(2) Series II with 5 turns of spiral: Flat bend, 3*d*, 6*d*, and 8*d* hooks.

(3) Series III with 7 turns of spiral: Flat bend, 3*d*, 6*d*, and 8*d* hooks.

(4) Series IV with 10 turns of spiral: Flat bend, 3*d*, 6*d*, and 8*d* hooks.

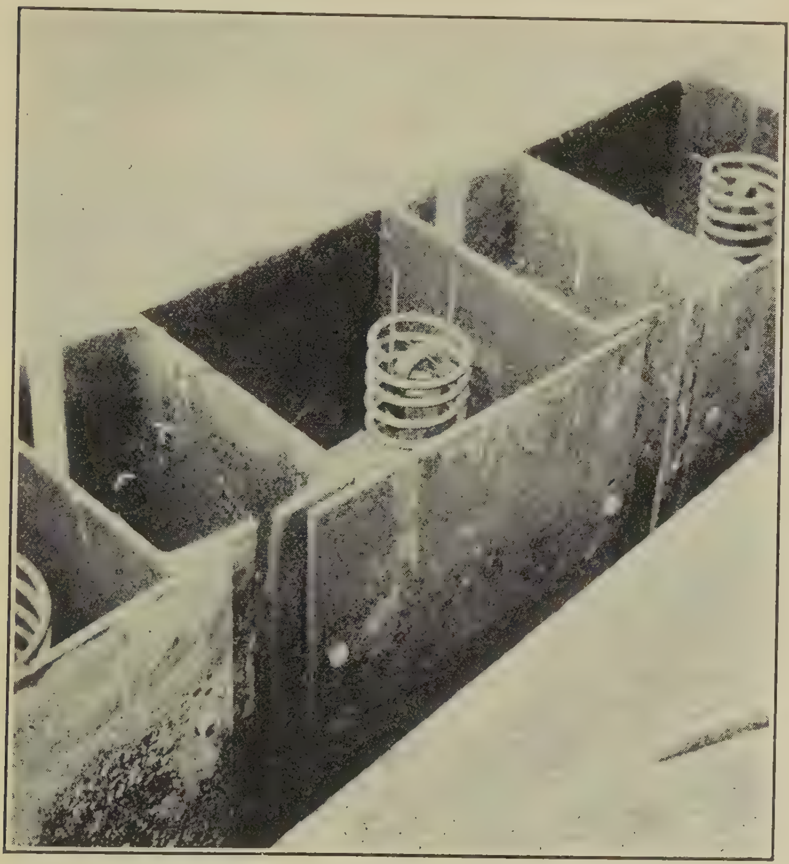


FIG. 9.—POSITION OF SPIRALS IN FORMS.

(5) Series V without spirals, a check on Series I: Flat bend, 3*d*, 6*d*, and 8*d* hooks.

(6) Special series without spirals: A quick check on the 12*d* specimen of Series I and on one intermediate size. 10*d* and 12*d* hooks cast in lumnite cement concrete.

The mix used in the first series was 1 part cement to 2.8 parts sand and 2.8 parts gravel, all by volume. All of the gravel passed the ¾-in. sieve, most of it being less than ½ in. in size.

The remaining series were proportioned by weight in order to keep a close control; slightly less than 7½ gal. of water per cu. ft. of cement were used, varying from time to time because of variations in the moisture content of the aggregates. In the specimens of the special series lumnite cement was used instead of portland cement, the specimens otherwise being identical. The control cylinders were made of concrete from the same batch as was used in the top portion of the specimen, that is, from the concrete enclosing the hooks.

The intention was to have concrete with a cylinder strength of 2,000 lb. per sq. in. Unfortunately, owing to Mr. Posey's illness, three of the specimens of Series I were given a longer period of setting than the others. The two specimens made of lumnite cement were tested at the age of 24 hours. Even then the concrete strength was much higher than the other specimens, and forms the basis for interesting comparisons later.

The spirals were placed by screwing them into the fresh concrete when it had been placed up to the level of the top of the hooks. Fig. 9 shows the position of the spirals in the forms. They were held there by a small wire, for the purpose of photographing only; no wires were used to hold the spirals in place when the specimens were cast. On dissection of the specimens some of the spirals were found to have sunk slightly out of place. This presumably occurred when the forms were being vibrated.

The specimens and control cylinders were cured in the damp room of the concrete laboratory of the Civil Engineering Department.

6. *Procedure.*—When ready to test, the specimens and cylinders were removed from the damp room. The specimen to be tested was hoisted into the specimen holder and accurately wedged in place. The grips were then adjusted and an initial load of 200 lb. applied. The dials were then attached. Fig. 10 shows the specimen and holder in place, on the testing machine, and Fig. 11 shows the specimen in place with the dials attached.

The load was then applied in 200-lb. increments at the slowest speed of the machine used, a 30,000-lb. Riehlé machine, the rate of application during the first stage of the test being about 1,000 lb. per minute. One man was required to operate the machine and another to read the dials and take notes. In some of the Series I tests, time was allowed for adjustment between loads, but in the testing of the remaining series increments were added as soon as the dials were read and the notes recorded. There was an appreciable slip if the loads were left on the specimen, so that a constant rate of testing was necessary if two different tests were to give comparable results. This slipping will be discussed later.

After slipping became general, or the stress in the bar had passed the elastic limit, the machine was shifted to a higher speed so that the load might again be raised. It was found that one series could be tested in a day. The cylinders were capped either with plaster of paris or with

neat cement, and were tested in the usual manner to determine the ultimate compressive strength.

The specimens for determining the friction of the sheath were tested in the same manner as the other specimens, the holders first being removed from the top of the machine.

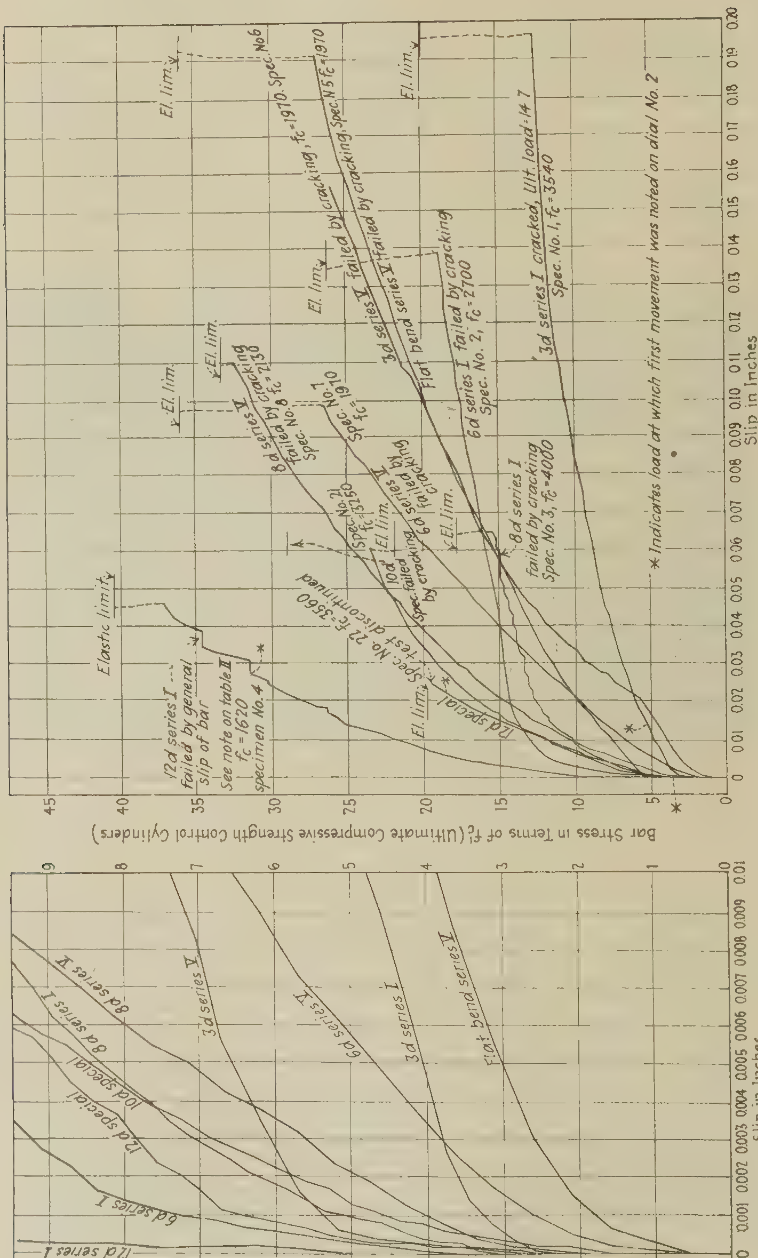
7. *Reduction of Data.*—The value of an anchorage depends partly upon the adhesive strength of the concrete and partly upon its compressive strength. Since, however, the adhesive strength and the compressive strength are directly proportional, within say 10 per cent to 15 per cent,

FIG. 10.—(LEFT) SPECIMEN AND HOLDER IN PLACE: FIG. 11 (CENTER) SPECIMEN IN PLACE WITH DIALS ATTACHED: FIG. 12 (RIGHT) FAILURE WITH SHORT-RADIUS HOOK.

a direct comparison of the values of the anchors tested has been made by stating the loads in terms of the ultimate compressive strength of the control cylinders. These strengths are given in several places in the accompanying tables, so that all values may be translated again into pounds if desired.

A complete log of the readings of Dials No. 1 is given in graphical form in Plates I to V, and upon these plates are also given certain other significant data, as the elastic limit of the steel, the beginning of slip of Dial No. 2, etc. To save space the readings for Dials No. 2 are not plotted, but the readings at several important stages are given in Tables II to IV. Where it would be difficult to distinguish between the curves on any plate the specimens have been identified in several places to avoid confusion.

Table II gives conditions at failure. Since, however, these conditions were obtained with the bars "on the run," they should be interpreted with



Bar Stress in Terms of f'c (Ultimate Compressive Strength Control Cylinders)
Slip in Inches
Elastic limit
El. lim.
12d series I failed by general slip of bar
See note on table II
fc=1620
specimen No. 4
12d special
Spec. No. 22 fc=3560 test discontinued
8d series V failed by cracking Spec. No. 8 fc=2130
Spec. No. 1 fc=1910
10d Spec. failed by cracking
6d series V failed by cracking
3d series V failed by cracking, fc=1970. Spec. No. 6
3d series V failed by cracking, Spec. No. 5 fc=1970
Flat bend series V failed by cracking
6d series I failed by cracking Spec. No. 2, fc=2700
8d series I failed by cracking Spec. No. 3, fc=4000
3d series I cracked, Ult. load=14.7 Spec. No. 1, fc=3540
* Indicates load at which first movement was noted on dial No. 2
12d series I
6d series I
8d series I
8d series V
10d special
3d series V
6d series V
3d series I
Flat bend series V

PLATE I

great caution. Table III indicates the condition when movement was first detected at Dial I, and Table IV when movement was first observed at

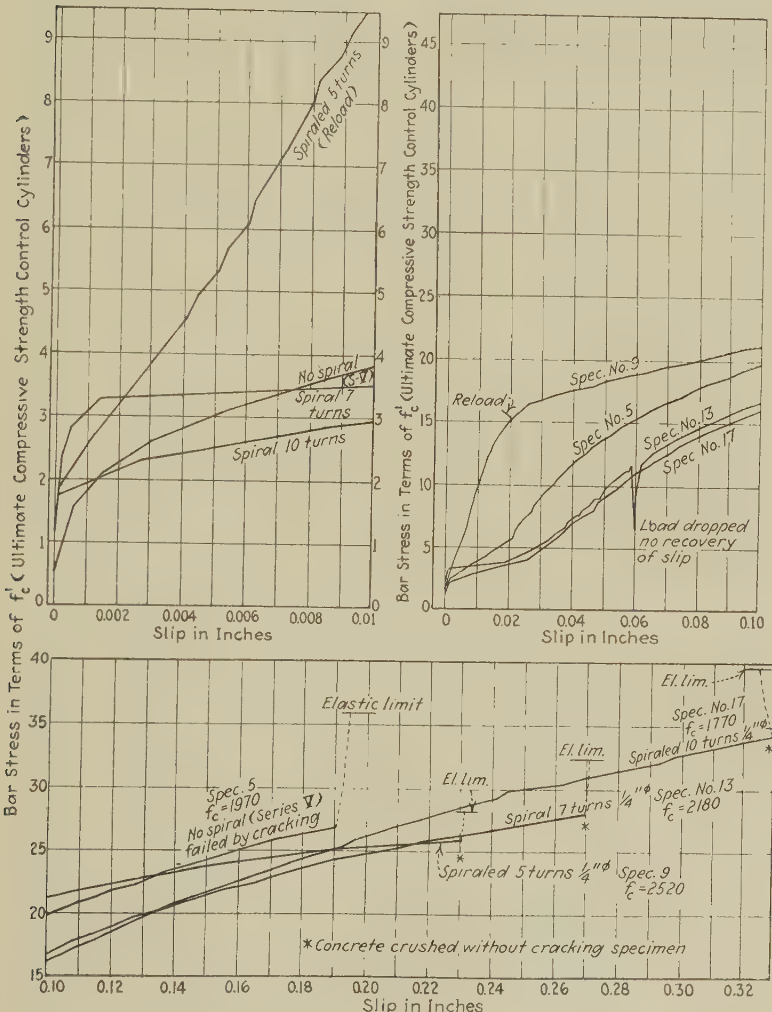


PLATE II.

Dial 2. It will be noticed from Table IV that the first movement at the end of all the bars was upward—around the bend; but in the case of the 3*d* hooks Table II shows that later the hook moved bodily downward, probably because of its relative stiffness. See also the remarks in Table IV.

TABLE II.—CONDITIONS AT FAILURE.

Series	Spec.	Hook Diameter	Spiral Turns	Cyl. f_c, lb. per sq. in.	Load, lb.	f_s, lb. per sq. in.	f_s/f_c	Cause of Failure	Dial No. 1, in.	Dial No. 2, in.	Nominal Bond, lb. per sq. in.	Nominal Bearing, lb. per sq. in.	Rate of Dial No. 1 over Dial No. 2, times as fast	Remarks
I	1	3d	..	3540	10,200	52,000	14.7	cracked	0.310 ↓	0.0145 ↓	947	14,650	10	
	2	6d	..	2700	10,000	51,000	18.9	"	0.139 ↓	0.079 ↑	692	8,000	1.7	
	3	8d	..	4000	12,000+	61,200	15.3	"	0.196 ↓	0.085 ↑	809+	6,860+	2.5	Dial Readings taken "on run" at higher load.
	4	12d	..	1620+	11,800+	60,200	37.1	stopped	0.143 ↓	0.050 ↑	540+	4,290+	1.6	Dial Readings taken at much higher load "on run."
V	5	1d	..	1970	10,400	54,100	26.9	cracked	0.191 ↓	....	1250	53,000	....	
	6	3d	..	1970	10,000	51,000	25.9	"	0.157 ↓	....	928	14,370	....	
	7	6d	..	1970	10,200	52,000	26.4	"	0.098 ↓	....	705	8,160	....	
	8	8d	..	2130	13,600	69,300	32.4	"	0.110 ↓	....	804	7,760	....	
II	9	1d	5	2520	13,600	69,300	26.0	"	0.230 ↓	....	1635	69,300	....	Load of 8,000 accidentally at start. Released and restarted.
	10	3d	5	2520	13,600	69,300	27.5	"	0.423 ↓	0.046 ↓	1263	19,550	6.0	Readings at 13,600 pounds. Failure at 14,500 pounds.
	11	6d	5	2520	18,000	91,800	36.4	slip	at 18,000 0.400 ↓	at 16,000 0.056 ↑	1246	14,400	2.5	Failure at 18,500 pounds.
	12	8d	5	2520	14,000	71,400	28.3	stopped	0.178 ↓	0.024 ↑	827	8,000	3.0	
III	13	1d	7	2180	12,000	61,200	28.0	broke at wire	0.269 ↓	....	1443	61,200	....	Failure at 13,000 pounds.
	14	3d	7	2180	12,000	61,200	28.0	crack	0.446 ↓	....	1113	17,250	....	See last par., Art. 9.
	15	6d	7	2180	15,000	76,500	36.0	crack	0.300 ↓	0.0007 ↑	1038	12,000	200	Readings at 15,000 pounds. Failure at 17,000 pounds.
	16	8d	7	2180	17,500	89,300	40.9	bar broke	0.220 ↓ at 17,500	at 13,800 0.0014 ↑	1034	10,000	90	Failure at 20,700 pounds.
IV	17	1d	10	1770	12,000	61,200	34.5	broke at wire	0.330 ↓	....	1443	61,200	....	Failure at 13,700 pounds.
	18	3d	10	2250	11,000	56,100	24.3	crack	0.365 ↓	0.0013 ↓	1021	15,800	160	Failure at 19,375 pounds.
	19	6d	10	2250	13,600	69,300	30.8	broke at wire	0.135 ↓	0.042 ↑	941	10,900	1.9	
	20	8d	10	1760	16,400	83,600	47.5	slip	0.137 ↓	failed to operate	970	9,360	....	
Sp.	21	10d	..	3250	15,000	76,500	23.5	crack	0.061 ↓	0.0225 ↑	773	6,660	1.5	Failure at 18,500 pounds.
	22	12d	..	3560	13,600	69,300	19.5	stopped	0.024 ↓	0.0003 ↑	622+	4,940+	11	

The specimens in the present investigation were tested to destruction without waiting for any time adjustment between loads, except for a few cases with the first series tested. The question naturally arises as to what would be the effect on the load slip curve if every load were left on until there was no further slip of the bar. It would be difficult to test a specimen in this manner. The slips from time effect were so large that the load could only have been held in an ordinary testing machine by means of springs. In all but one of the few cases where loads were held for a time, the effects have been eliminated from the accompanying plates. This effect is shown in specimen 2, Plate IV, near the beginning of the curve. By eliminating all pauses in loading one basis of comparison of relative values is obtained. It would have been better, no doubt, if equal increments of load were added in equal increments of time, but the rate of loading was dependent on how fast each specimen would pick up load from a machine driven at constant speed. Even if such a load-slip curve were obtained, there would still be the question of the effect of repeated applications of load, this being the condition of loading of a beam in reinforced-concrete construction. The effect is, of course, much more marked toward the ultimate loads, and in most cases it was necessary to shift to the next higher speed of the machine to lift the scale beam at all. These facts must not be overlooked in making comparisons.

TABLE III.—CONDITIONS AT FIRST SLIP OF DIAL NO. 1.

Series	Spec.	Hook Diameter	Spiral Turns	Load, lb.	f_s lb. per sq. in.	f_s/f_c	Nominal Bond, lb. per sq. in.	Nominal Bearing, lb. per sq. in.	Slip Dial No. 1, in.
I	1	3d	..	1,800	9,200	2.59	167	3,600	0.0001
	2	6d	..	2,200	11,200	4.15	152	1,760	0.0001
	3	8d	..	2,600	13,300	3.31	154	1,485	0.0001
	4	12d	..	1,800	9,200	5.65	79	654	0.0001
V	5	1d	..	400	2,000	1.04	48	2,040	0.0003
	6	3d	..	1,200	6,100	3.11	111	2,400	0.0001
	7	6d	..	400	2,000	1.04	28	320	0.0001
	8	8d	..	1,000	5,100	2.39	59	571	0.0001
II	9	1d	5	1,000	5,100	1.92	120	5,100	0.0001
	10	3d	5	800	4,100	1.62	74	1,600	0.0003
	11	6d	5	1,600	8,200	3.23	111	1,280	0.0001
	12	8d	5	1,200	6,100	2.43	71	686	0.0001
III	13	1d	7	800	4,100	1.87	96	4,100	0.0002
	14	3d	7	800	4,100	1.87	74	1,600	0.0001
	15	6d	7	400	2,000	0.93	28	320*	0.0008*
	16	8d	7	1,200	6,100	2.80	71	686	0.0001
IV	17	1d	10	600	3,100	1.73	72	3,100	0.0001
	18	3d	10	400	2,000	0.88	37	800*	0.0007*
	19	6d	10	400	2,000	0.91	28	320*	0.0005*
	20	8d	10	900	4,600	2.60	53	513	0.0003
Sp.	21	10d	..	1,200	6,100	1.88	62	533	0.0001
	22	12d	..	2,200	11,200	3.15	101	800	0.0001

* Large movement at low load probably due to slightly defective concrete under hook.

In Tables II to IV nominal bond and bearing values are given. In determining the nominal bond the whole exposed area (except the end of

the bar) was used. It is evident that in the case of flat bends this must be a questionable method of computation. Similarly, when the 3*d* hooks moved bodily downward it is evident that the concrete was not in contact with the steel on the outside of the bend, and hence the nominal bond

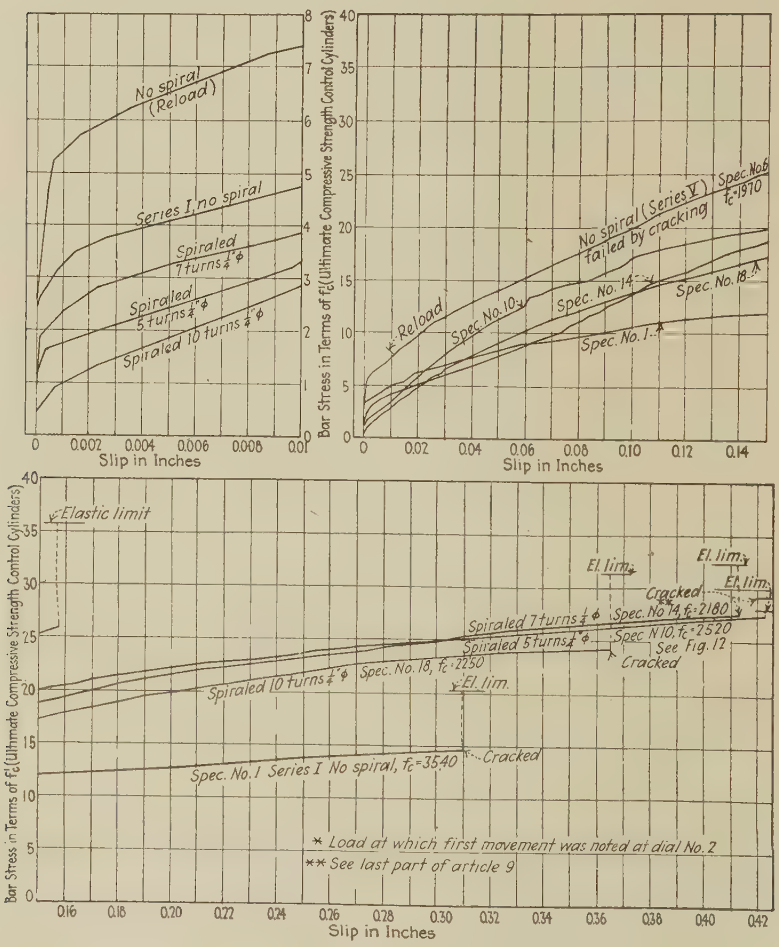


PLATE III.

cannot represent even the true average. Since the larger bends slid around the curve, the nominal bond more nearly represents the true average, although the contact on the outside of the bend might still be open to question.

In computing the bearing under the bend, the area of the end of the bar was included in the flat bends and in the 3*d* bends after these

latter had moved back down to their original position. Toward failure the flat ends bent backwards in the form of a knuckle so that a considerable and undetermined portion of the load was carried in this manner. The effect, however, was the same as if the bearing had taken place under the bend. The value given in the Table is based on the assumption that all the load on the flat bends was carried on the end only of the bar, and

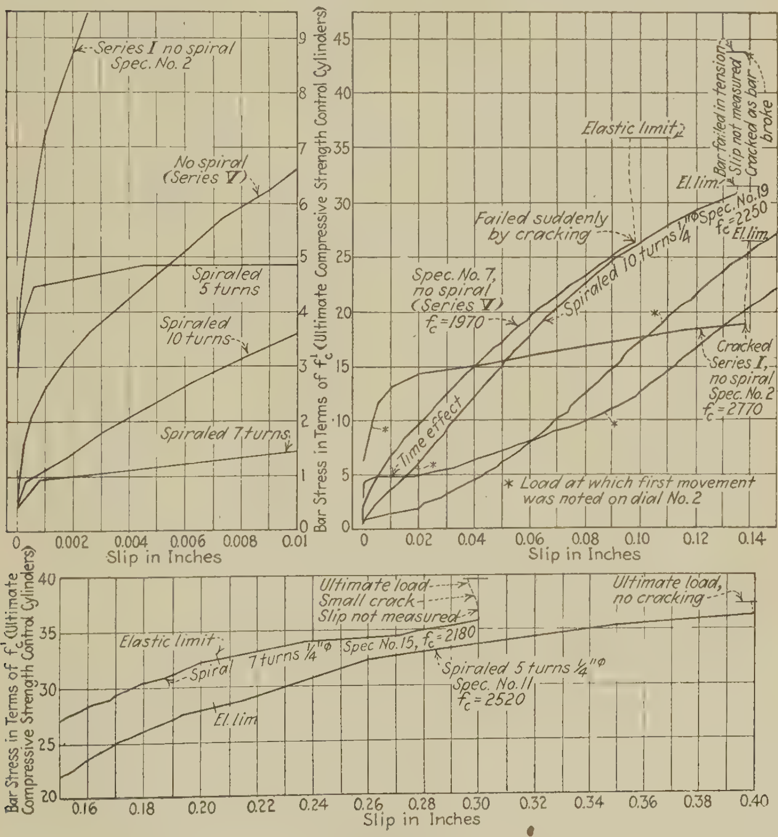


PLATE IV.

hence is too large. The $3d$ hooks also exhibited this tendency to bend backwards at higher loads, which is consistent with the fact that they moved bodily downward, thus inducing a pressure under the points.

8. *Manner of Failure.*—It is of interest to note from Table II that in the unspiraled specimens failure was due in each case to splitting, except with the $12d$ hooks. Most of the specimens, though not all, cracked suddenly. One of the specimens, the $10d$ hook cast in lumnite cement, ex-

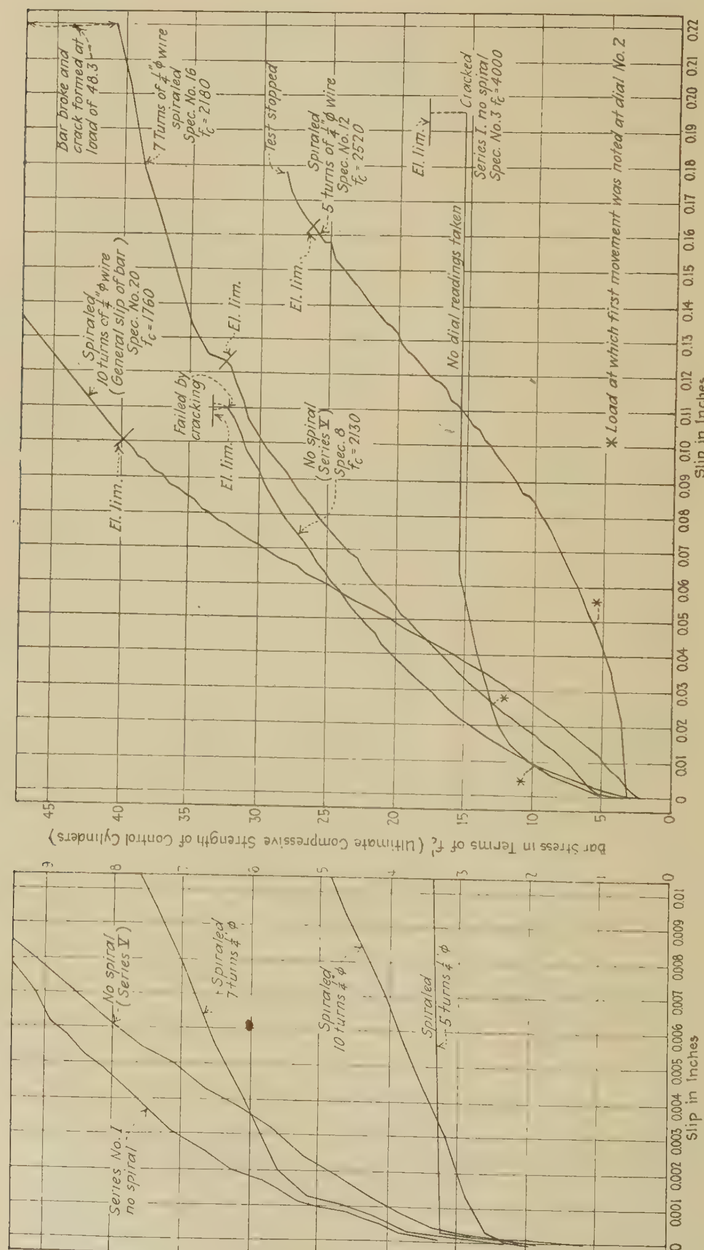

Bar broke and crack formed at load of 48.3
7 Turns of $\frac{1}{4}$" ϕ wire spiraled Spec. No. 16 f_c' = 2180
Test stopped
Spiraled 5 turns of $\frac{1}{4}$" ϕ wire Spec. No. 12 f_c' = 2520
El. lim.
Cracked
Series I. no spiral Spec. No. 3 f_c' = 4000
Spiraled 10 turns of $\frac{1}{4}$" ϕ wire (General slip of bar) Spec. No. 20 f_c' = 1760
Failed by cracking
No spiral (Series V) Spec. 8 f_c' = 2130
No dial readings taken
* Load at which first movement was noted at dial No. 2
Slip in Inches
Bar Stress in Terms of f_c' (Ultimate Compressive Strength of Control Cylinders)
No spiral (Series V)
Spiraled 7 turns $\frac{1}{4}$ ϕ
Spiraled 10 turns $\frac{1}{4}$ ϕ
Spiraled 5 turns $\frac{1}{4}$ ϕ
Series No. I no spiral
Slip in Inches

PLATE V.

hibited a particularly violent failure, almost dislodging the whole apparatus from the machine. Having been made cautious by this experience, the test of the special 12*d* hook, unspiraled, was not carried to failure.

The manner of failure of the specimens with spiraled hooks was different. No cracks appeared on the surface of the specimens with flat bends. The 3*d* hooks, with the same size spiral as the flat bends, split the concrete in every case. This spiral was so small that the eccentric load put on it by the 3*d* hook caused the block of concrete enclosed by the

TABLE IV.—CONDITIONS AT FIRST SLIP OF DIAL NO. 2.

Series	Spec.	Hook Diameter	Spiral Turns	Load, lb.	f_s, lb. per sq. in.	f_s/f_c	Dial No. 2, in.	Dial No. 1, in.	Nominal Bond, lb. per sq. in.	Nominal Bearing, lb. per sq. in.	Rate of Slip Dial No. 1 over Dial No. 2
I	1	3d	..	4,000	20,400	5.76	0.0001 ↑	0.0180	......	......	*
	2	6d	..	5,200	26,500	9.81	0.0001 ↑	0.0028	......	......	6
	3	8d	..	7,800	39,800	9.93	0.0002 ↑	0.0091	......	......	5
	4	12d	..	10,600	54,100	33.3	0.0001 ↑	0.0330	......	......	10
V	5	1d	..	......	......	.....	....	......	......	......	...
	6	3d	..	......	......	.....	....	......	......	......	...
	7	6d	..	......	......	.....	....	......	......	......	...
	8	8d	..	......	......	.....	....	......	......	......	...
II	9	1d	5	......	......	.....	....	......	......	......	...
	10	3d	5	2,600	13,300	5.26	0.0001 ↑	0.0175	......	......	†
	11	6d	5	5,200	26,500	10.51	0.0005 ↑	0.0860	......	......	16
	12	8d	5	2,800	14,300	5.66	0.0001 ↑	0.0490	......	......	60
III	13	1d	7	......	......	.....	....	......	......	......	...
	14	3d	7	......	......	.....	....	......	......	......	...
	15	6d	7	8,000	40,800	18.68	0.0001 ↑	0.1070	......	......	180
	16	8d	7	5,400	27,500	12.61	0.0001 ↑	0.0253	......	......	42
IV	18	1d	10	......	......	.....	....	......	......	......	...
	18	3d	10	3,000	15,300	6.62	0.0001 ↑	0.0305	......	......	‡
	19	6d	10	2,400	12,200	5.43	0.0001 ↑	0.0180	......	......	17
	20	8d	10	failed to operate	......	.....	....	......	......	......	...
Sp.	21	10d	..	6,600	33,700	10.36	0.0001 ↑	0.0078	......	......	6
	22	12d	..	13,000	66,300	18.59	0.0001 ↑	0.0218	......	......	15

* Dial No. 1 moved 0.061 in. while Dial No. 2 moved 0.0004 in. up and back. Dial No. 1 then moved 50 times as fast as Dial 2, decreasing to 10 times. Average = 16.2.

† Dial No. 1 moved 0.0385 in. while Dial No. 2 moved 0.0004 in. up and back. Dial No. 1 then moved 10 times as fast as Dial 2, decreasing to 6 times. Average = 8.0.

‡ Dial No. 1 moved 0.10 in. while Dial No. 2 moved 0.0003 in. up and back. Dial No. 1 then moved 70 times as fast as Dial 2, increasing to 160 times. Average = 125 times.

spiral to split bodily out of the specimen (after the concrete inside the spiral was crushed under the bend, however). A characteristic failure of this type is shown in Fig. 12. It is noteworthy that Saliger's specimens, in which hooks of short radius of bend were enclosed in small blocks of concrete, were not liable to failure due to this cause, since the specimen was tightly wedged in the holder. (See Fig. 1.) The flat bends did not show a failure of this type because compressive failure occurred at a load lower than that required to split the spiral from the rest of the specimen. The 6*d* and 8*d* hooks split the concrete in some cases, but not in others. Always, however, failure was accomplished by a large slip.

The failure of the spiraled specimens, even when due to cracking, was not violent, and the loads were carried as high as possible. In four cases, with the 7 and 10 turns of spiral, the bar actually broke, although in three of these cases the break occurred at the tiny hole where the measuring wire was inserted.

9. *Discussion of Data.*—The nominal values of u is given in Table II vary from a magnitude of fictitious height for flat bends to a value for 12 hooks not greatly larger than would be found in pull-out tests of straight bars. This would indicate that as the diameter of the hook increased the stiffness of the bar was of less significance, and that in the larger specimens rope-friction action was being developed. The surprising feature is the magnitude of the bearing stress under the hook. For the $3d$ and flat hooks the backward bending gives to these values an undue prominence, but even with the large hooks very large values were obtained.

In the case of all hooks this question of bearing comes up, which is the difference between a hook and a straight extension of equal length. Even if the hook were as flexible as a piece of rope, the tendency to cut in is measured by the pull on the bar. The high bearing values developed indicate that any anchor which relies on the bearing value alone of the concrete is bound to be a good one, but splitting tendencies must be entirely eliminated.

It will be noted both in Table II and Table IV that the rate of movement of the bar at the point of tangency is very much faster than that at the end of the hook. Table IV shows, too, that a movement of from 0.01 in. to 0.10 in. had taken place at the point of tangency before any movement was discernible at the end. This indicates that from the very beginning to the ultimate load compression of the concrete was in progress. The relative absolute movement indicated by the two dials varied considerably, but in no case did that of Dial No. 2 reach 0.01 in. before failure. This would indicate that the short straight portion at the end of the hook had but little influence.

Table IV is of interest also in showing that by the time the point of tangency had moved 0.01 in. the whole hook was in motion. It is questionable whether loads higher than those recorded in this Table could have been maintained indefinitely. Table II, owing to the fact that values were obtained "on the run", should not be taken very seriously except in a pictorial sense.

The curves on Plates I to V show not only how rapid was the movement of the bars, but indicate a curious property of the spiraled hooks. Without exception during the early loads the movement was much greater than in the unspiraled specimens. This can be explained on no other grounds than that the concrete under the bends was in some way affected, even with the great care taken in casting the specimens. As a device to improve the carrying capacity of hooks spirals apparently have an advantage only at high loads, when slip has already rendered the anchorage practically useless. The comparative loads carried at first slip, shown in Table III, bring out the same point.

The care required in the placement of the spirals is well brought out in Specimen 14. For some reason or other the spiral used in this specimen sank about 2 in. in the concrete. During the test a crack formed near the top, in the plane of the hook, followed the hook down to the spiral and then passed around outside the spiral. The slip at failure was nearly ½ in., although the load was not unsually high.

10. *Conclusions.*—(1) It is evident from this investigation that flat bends, spiraled or unspiraled, do not form a satisfactory anchorage. It will be noted that no flat bend developed the elastic limit of the steel.

(2) Hooks 3*d* in diameter do not behave much better, although there is still the possibility that if the straight piece at the end were extended for say 3 to 6 diameters further and enclosed in a long spiral in the nature of a long sleeve, the breaking out of the whole spiral might be prevented. Only one 3*d* hook, with 5 turns of spiral, in 2,500-lb. concrete, developed the elastic limit of the steel.

(3) All 6*d* and 8*d* spiraled hooks developed the elastic limit of the steel and one 8*d* spiraled hook in 2,200-lb. concrete developed the ultimate strength of the steel.

(4) The 10*d* and 12*d* hooks, unspiraled, in 3,200-lb. concrete, developend the elastic limit of the steel with a slip of 0.05 in.

(5) Hence, increasing the diameter of the hook increases the load-carrying capacity, and spiraled hooks showed greatest strength, but it is hard to see any direct relationship.

(6) The strength of the concrete certainly has some influence, although with the exception of Specimens 21 and 22 the concrete strength in this investigation did not vary greatly. There is some question, owing to difficulties in the curing room, whether the cylinders of Series I correctly represent the strength of the specimens.

(7) The size of the specimen to be split certainly has something to do with the strength of the unspiraled anchor. In this investigation, regardless of hook diameter, all unspiraled specimens were split, even though the depth of the block was 18 times the bar diameter.

(8) The size of the specimen had little to do with the strength of the spiraled specimens, for hooks of 6*d* or greater either crushed the concrete or pulled around the bend.

(9) The strength of the 12*d* hooks was in part due to the strength of the concrete and in part due to the comparatively easy bend. It is unfortunate that only one of these tests was carried to failure.

(10) Slip was evident at the point of tangency from very low loads, and movement of the whole bar at loads far below the ultimate strength developed. Since the prevention of slip is an important function of an anchor, hooks have still to demonstrate their right to be styled "adequate" anchors. The facts must not be overlooked that the values obtained in this investigation were comparative only, and no time was given for the very evident tendency for "time-slip."

Mr. Larson.

Louis J. Larson.—This paper adds much to the rather meager supply of information on the subject. The results show the loads that can be carried by hooks of various diameters using ½ in. bars and the type of failure that is likely to occur. However, the values of "nominal bond" and "nominal bearing" differ so much from each other and from the values obtained in other kinds of tests that the question must be raised, "What do they really mean?"

Most of the computed bond stresses and the bearing stresses for the flat bends are abnormally high. If the stresses obtained vary so greatly for various sizes of hooks what assurance is there that these results will apply to bars of another diameter? If the data can be interpreted in some rational manner so that the results become consistent among themselves and agree with the bearing and bond strength of concrete found in other tests, there is a greater likelihood that the results will apply to other conditions. This will greatly enhance the value of these tests.

While the general action of a hook embedded in concrete may be visualized it is difficult to analize the behavior. The pull-out resistance or strength of a hook is dependent upon several factors among which are the stiffness of the bar, friction between the bar and the concrete due to pressure set up by the tension on the bar, and bond on the embedded portion of the bar.

The strength developed by the stiffness of the bar is equal not alone to the strength of a hook of a certain size fitting around a cylinder. A considerable additional pull is required to bend the bar at the ends of the arc. Since the stiffness of a bar varies with the fourth power of the diameter it is probable that the contribution due to the stiffness becomes greater as the size of the bar increases even though the ratio of the diameter of the hook to the diameter of the bar remains constant.

The frictional resistance to motion between the bar and the concrete undoubtedly accounts for a considerable proportion of the strength of the hook. A small force at the far or low-tension end of the hook will resist a considerable pull on the bar because of the rope friction developed. A resistance at any point on the hook is similarly increased, the amount of the increase depending upon the friction and distance from the bar side of the hook. The bond on the bar furnishes such resistance at various points along the hook.

To find the sum of the effects or the total tension which they will resist let u = bond in lb. per sq. in.

f = coefficient of friction between the concrete and steel

a = the total angle of the hook.

r = the radius of the hook.

o = the circumference of the bar.

In Fig. 1 consider the effect at A due to the bond on an elementary length of the rod dl

$$dl = rd\beta$$

The bond on the length $dl = uord\beta$ and the tension developed at A due to the bond on dl is

$$dT = ourd\beta\, e^{f\beta}$$

The total tension that may be developed at A is

$$T_1 = \int dT = \int_0^{\alpha} uoTd\beta e^{f\beta}$$

$$= uor \frac{e^{f\alpha} - 1}{f}$$

If to the hook there is added a straight portion of length l_1 the resistance developed at A due to the bond on this length is

$$T_2 = uol_1 e^{f\alpha}$$

The total tension at A then becomes

$$T_3 = uol_1\, e^{f\alpha} + uor \frac{e^{f\alpha} - 1}{f}$$

Due to a straight portion beyond A the bond strength will be uol_2 when l_2 is the embedded straight length. The total tension then becomes

$$T = uol_1\, e^{f\alpha} + uor \frac{e^{f\alpha} - 1}{f} + uol_2 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

If the values of u and f are known it is possible to compute the portion of the hook strength due to bond and friction.

The above expression gives the least tension that will produce a failure by slipping. If the bar is large or has a hook of small diameter the tension developed may be much larger than that computed above because of the stiffness of the hook. Such is certainly the case for a bar bent flat on itself (hook diameter = d). Failures may also occur at loads below those computed if the compression stresses are sufficient to cause splitting or bursting of the concrete.

If a value of the coefficient of friction is assumed, Eq. 1 may be applied to the author's tests to determine the average value of the bond developed. Rewriting the equation

$$T = u\,(ol_1 e^{f\alpha} + or \frac{e^{f\alpha} - 1}{f} + ol_2) = Ku$$

$$\text{and } u = \frac{T}{K} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$\text{where } K = (ol_1 e^{f\alpha} + or \frac{e^{f\alpha} - 1}{f} + ol_2)$$

In Table A the values in the first six columns are taken from the author's Table II except that the maximum recorded loads are shown. Col. 7 gives the values for K using the dimensions of the hooks and assuming a value of

$f = 0.3$. Col. 8 gives the bond stresses computed from Eq. 2. For the larger hooks the compressive stress in the concrete under the bends was computed by finding the radial component of the tension on the bar and assuming it is carried on an area as wide as the diameter of the bar.

The total reaction of the concrete against the bar is

$$2T \sin \frac{\Delta\beta}{2}$$

and the area of concrete in contact with the bar is $r\Delta\beta d$, (Fig. 2).

If S_c is the compressive stress in the concrete

$$S_c r\Delta\beta d = 2T \sin \frac{\Delta\beta}{2}$$

For small angles $\frac{\sin \Delta\beta}{\Delta\beta} = 1$

$$\therefore S_c = \frac{T}{rd}$$

Since $d = \frac{1}{2}$ in. for the bars used

$$S_c = 2\frac{T}{r} \qquad (3)$$

where r = radius of hook.

Values of S_c are given in Col. 10.

Col. 9 gives the ratio of the computed bond stresses to the cylinder strength of the concrete, and Col. 11 shows the ratio of the computed compressive stresses to the strength of the cylinders.

The bars having a hook diameter of d and 3d have been treated in a separate manner and will be discussed later. The computed values for these specimens are shown in Table B.

The bond stresses shown in Table A range from 334 lb. per sq. in. to 842 lb. per sq. in. The specimens which showed the low values for bond were stopped before failure occurred. If these tests are excluded the lowest bond stress is 433 lb. per sq. in. Since the strength of the concrete varied considerably a better comparison is furnished by the ratio of the bond stress developed to the cylinder strength of the concrete (given in Col. 9). This ratio varies from 10.8 to 37.5 per cent and most of the values lie between 20 and 35 per cent. This variation is no greater than that found by Abrams,* and the values agree with his results when allowance is made for the difference in strength of cubes and cylinders.

Specimens 2 to 4 show relatively low bond stresses but all these specimens failed by cracking of the concrete and there is no indication that the maximum bond was developed at failure. It should also be noted that the low ratios in this series compared with series V is due to the high strength of the cylinders for series I. The actual loads for the two series are about the same.

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* Abrams, University of Illinois Bulletin No. 71.

The action of a bar with a flat bend is quite different from that of a bar with a large hook. For a flat bend the action resembles that of a straight bar with a lug welded on one side. There can be no rope friction in this case and the increase in the holding power of a bar with a flat bend over that of a straight bar having the same bond area is due to the bearing of the bent-over end on the concrete. In this case it would be absurd to apply Eq. 2 to determine the bond stress or Eq. 3 to determine the compressive stress in the concrete. A flat-bend hook is statically indeterminate. Part of the total load

TABLE A

1	2	3	4	5	6	7	8	9	10	11
Specimen No.	Hook Diam.	Load in 1000 lb.	f_c	Cause of Failure	Dial No. 1	K	u lb. per sq. in.	u/f_c	S_c lb. per sq. in.	S_c/f_c
1	3d	10.2	3540	cracked	.310	16.90	...	....	....	....
2	6d	10.0	2700	cracked	.139	23.05	433	16.0	13.3	4.93
3	8d	12.0	4000	cracked	.196	27.15	442	11.0	12.0	3.00
4	12d	11.8	1620+	stopped	.143	35.35	334	20.6	7.9	4.88
5	1d	10.4	1970	cracked	.191	.....	...	....	....	....
6	3d	10.0	1970	cracked	.157	16.90	...	....	....	....
7	6d	10.2	1970	cracked	.098	23.05	443	22.5	13.6	6.90
8	8d	13.6	1970	cracked	.110	27.15	501	25.4	13.6	6.90
9	1d	13.6	2520	cracked	.230	.....	...	....	....	....
10	3d	14.5	2520	cracked	.423	16.90	...	....	....	....
11	6d	18.5	2520	slip	.400+	23.05	803	31.8	24.7	9.78
12	8d	14.0+	2520	stopped	.178	27.15	516	20.4	14.0	5.56
13	1d	13.0	2180	bar broke	.269+	.....	...	....	....	....
14	3d	12.0	2180	cracked	.446	16.90	...	....	....	....
15	6d	17.0	2180	cracked	.300+	23.05	738	33.9	22.6	10.37
16	8d	20.7	2180	bar broke	.220+	27.15	762	34.9	20.7	9.48
17	1d	13.7	1770	bar broke	.330+	.....	...	....	....	....
18	3d	11.0	2250	cracked	.365	16.90	...	....	....	....
19	6d	19.4	2250	bar broke	.135+	23.05	842	37.5	25.8	11.47
20	8d	16.4	1760	slip	.137	27.15	605	34.4	16.4	9.32
21	10d	18.5	3250	cracked	.061+	31.20	593	18.3	14.8	4.55
22	12d	13.6	3560	stopped	.024	35.35	385	10.8	9.1	2.54

TABLE B

Specimen No.	Hook Diam.	Load in 1000 lb.	Assumed						
			f_c	u/f_c	u	P_u	P_c	S_c	S_c/f_c
1	3d	10.2	3540	.30	1062	7.5	2.7	7.2	2.0
5	1d	10.4	1970	.30	591	4.2	6.2	24.8	12.6
6	3d	10.0	1970	.30	591	4.2	5.8	15.4	7.8
9	1d	13.6	2520	.30	756	5.3	8.3	33.2	13.2
10	3d	14.5	2520	.30	756	5.3	9.2	24.6	9.8
13	1d	13.0	2180	.30	654	4.7	8.3	33.2	15.2
14	3d	12.0	2180	.30	654	4.7	7.3	19.4	9.0
17	1d	13.7	1770	.30	531	3.7	10.0	40.0	22.6
18	3d	11.0	2250	.30	675	4.8	6.2	16.6	7.4

on the bar is carried by bond on the straight, embedded portion and the remainder is carried in bearing on the concrete. These two actions are independent of each other. The load carried by the bond stresses does not produce a bearing stress under the projection and vice-versa. This is quite different from the action of the large hooks in which the bond stresses and the bearing stresses act simultaneously, and one stress does not relieve the other. The bearing and the bond stresses are each a function of the total load for the large hooks.

For a flat bend the area subjected to the bearing stress is not only the cross section of the end of the bar but also the area of the concrete under the bend. Hence the compressive area is ¼ sq. in. instead of the area of a ½ in. round bar. The difference between these two areas amounts to 25 per cent.

A 3d hook may act, to a large extent, like a flat bend. The dial readings show that the whole bar moved downward at the ultimate load and hence there could be no rope friction around the bend. The bearing area in this case is ½ x 1½ in. but the bearing pressure is not uniformly distributed as shown by the fact that dial No. 1 moved from 100 to 300 times as far as dial No. 2. In the absence of definite information as to the distribution of the bearing pressure it may perhaps be assumed without much error that the pressure varies uniformly from a maximum at dial 1 to zero at dial 2. The maximum bearing stress will then be twice the average.

Without exact knowledge as to the deformations it is not possible to compute the proportion of the load carried by the bond stresses and that carried by bearing. However, it is possible to determine whether or not the total load carried can be accounted for by stresses of a reasonable magnitude. An arbitrary division of the load may be assumed and the stresses computed on that basis, or one of the stresses may be assumed and the other computed. In Table B, a bond stress of 30 per cent of the cylinder strength of the concrete was assumed to act on the straight portions of the bar. The load accounted for by this bond stress was deducted from the total load, and the bearing stress required to carry the remainder of the load is given.

The compressive stresses given in Table A were obtained by the use of Eq. 3. This equation is based on the assumption that the radial component of the tension in the bar at any section is held in equilibrium by the compressive stresses in the concrete. For a bar having no stiffness or for a bar with a uniform tension this assumption is correct. If the tension in the bar varies from section to section, the beam action of the bar may distribute the compressive stresses somewhat and thus reduce the maximum. However, for the larger hooks the tension does not diminish rapidly and the reduction in compressive stresses from this source cannot be large. Hence stresses approximately as high as the values shown in Table A must have been developed.

The specimens with spirals developed bearing stresses of from 9.3 to 11.5 times the cylinder strength of the concrete with the exception of specimen No. 12 on which the test was stopped. In series V the bearing stresses are about 7 times the cylinder strength of the concrete and in series I the loads carried are about the same as series V but the cylinder strengths are much higher, resulting in lower ratios.

In Table B, the ratios of computed bearing stress to the cylinder strength of the concrete for the 3d hooks are fairly consistent and agree with results shown in Table A, except specimen No. 1. The low ratio for this specimen is again accounted for by the high cylinder strength. The compressive stresses obtained for the 1d hooks are very high. Is it not possible, however, that such stresses may be developed on an area of ¼ sq. in. completely surrounded by a mass of concrete? The greater part of the compressive area in this case is perpendicular to the load. For all other hooks the bearing is on the side of a round bar and there must be considerable splitting tendency. Mr. Mylrea points out that for the 3d hooks the spiral split bodily from the specimen and observes that the flat bends did not fail in this way because a compression failure occurred at a load lower than that required to split the spiral from the rest of the specimen. This conclusion is not evident from Table II. For two of the three series having spiral reinforcement, the load carried by the flat bends was greater than that carried by the 3d hooks. The splitting out of the spiral is more likely due to the greater eccentricity of the load on the 3d hooks.

A study of the values in Tables A and B shows that on the basis of the foregoing analysis it is possible to account for the loads carried by the hooks without any unusual values of bond stresses. The higher values for the hooks of 6d and larger diameter, are well within the limits found by Mr. Abrams. The lower values were all found for specimens which failed in some other manner and there is no indication that bond failure was imminent. For the flat bends and 3d hooks a conservative value for the bond stress was assumed and the resulting compressive stresses computed. If higher bond stresses were developed, the compressive stresses necessary for equilibrium were lower than the computed values.

The computed compressive stresses are relatively high but quite consistent for each type of specimen, spiraled or unspiraled. The low values are found for specimens which did not fail by cracking. The apparently abnormal values developed in the flat bend specimens may be due to favorable conditions of the bearing surface. The writer is not aware of any bearing tests made under similar conditions, but a bearing stress of even 40,000 lb. per sq. in. does not appear impossible if applied on a small area surrounded by a mass of concrete.

If the analysis presented is correct it becomes a simple matter to compute the probable strength of a hook. However, in the derivation of the equations certain assumptions were made which need verification by further tests. Hooks of more or less than 180-deg. bends would be helpful. Load-slip curves could also be made on flat-bend specimens in which either the bearing or the bond is eliminated to determine the load carried by each. Such tests would indicate a reasonable division of the load between bond and bearing stresses.

The size of hook necessary to obtain rope-friction action is an open question. In Table B the 3d bend was treated like a flat bend because the whole hook moved downward at the ultimate load. At lower loads these bars showed an upward movement at dial No. 2 and rope friction probably

played a part in the early stages. As a matter of fact very reasonable values are obtained for the stresses at failure applying Eqs. 1 and 3 to the 3d hooks. For larger bars the size of hook required for rope friction may be even greater than for these tests, due to the fact that the stiffness of the bar increases with the fourth power of the diameter whereas the load increases with the square and the moment arm with the first power making the moment increase with the cube of the diameter of the bar.

The author states that the small movement at dial No. 2 indicates that the straight portion at the end of the hook had very little influence. From Table II it is seen that dial No. 2 showed movements as large as .085 in. in one case. Mr. Abrams found that the maximum bond was developed at an end slip of .01. Although the movement at dial No. 2, does not represent the slip at the end of the bar, the readings indicate that considerable bond must have been developed on the straight portion beyond the hook. The bond on this portion is multiplied by about 2.5 at the other end of the hook. According to Eq. 1 this straight piece carries from 17 to 36 per cent of the total load for the hooks used.

Mr. Mylrea emphasizes the danger of compressive stresses and the necessity of eliminating them. The large deformations and the type of failures obtained indicate the danger of high bearing stresses. A large hook will reduce the bearing stress developed, but there is often insufficient room for such a hook.

Would it not be feasible to make a hook with a varying radius, similar to a spiral? It would be possible to make a hook with a 12d diameter at the starting end and decrease the radius, either gradually or by steps, to 3d with a straight portion at the end. Such a hook could be contained in the space required for a 6d hook and it should be as strong as a 12d hook. A specimen with such a hook can readily be made up in the laboratory and tested. If the results indicate the desired advantage, means for bending them on a practicable scale should be readily devised.

Mr. Lindau. CHAIRMAN LINDAU.—May I ask the size of the bars in those beams?

Mr. Mylrea. T. D. MYLREA.—They were nominally ½-in. round, but they were slightly oversize, so that the twenty bars gave us an area of almost exactly 5 sq. in.

Mr. Slater. W. A. SLATER.—Since the spiral hooks did not show as good results as might be expected, would Professor Mylrea consider that a spiral hook will add to the carrying capacity of the beam above what a hook without a spiral will add?

Mr. Mylrea. T. D. MYLREA.—In spite of the fact that the spiral hooks did not show as great strength in the early loads, I think that spirals would add to the ultimate capacity of the member, because without the spirals the chances are that the beams would be split long before the ultimate carrying capacity of the hooks would be reached.

Mr. Posey, Jr. C. J. POSEY, JR.—I performed some of the experimental work with Professor Mylrea, and can reply to Professor Slater's question. While the spirals helped the small hooks considerably, so far as ultimate load was concerned, they did not help out the slips at low loads. Also we found

that the very large hooks were so large that spirals did not help them very much. That is easily understood, because with a large hook the slipping forces are distributed over a larger area of concrete. Further, they are not so likely to cause local overstress, and, therefore, form a better protection against a slipping tendency than would small hooks reinforced with spirals. The stress in the spiral is rather high and the area of the steel is small, so that there is a comparatively large deformation compared with what would result from a small stress and a comparatively large block of concrete; it is these deformations that cause the slip in the hooks.

An inspection of the load-slip curves of the different specimens shows that the larger hooks and particularly the 12d hook, carry the greatest load with the least slip over the vital range of values for anchorages in beams. The splitting force exerted by the 12d hooks is evenly distributed over such a large area that they are not likely to split the concrete. In the specimen tested to failure, the block although very little larger than the specimen and entirely unrestrained, was not split by a (relative) bar stress higher than that of any other unreinforced specimen tested. It would be interesting to compare the load-slip curve of a 12d hook with that of a straight embedment of the same length, slip being measured at the loaded end. The maximum bond stress developed, equivalent to 650 lb. for 2000-lb. concrete, would seem to indicate that the 12d hook would be the better.

I do not understand the author's last conclusion wherein he states, "Slip was evident at the point of tangency from very low loads, and movement of the whole bar at loads far below the ultimate strength developed. Since the prevention of slip is an important function of an anchor, hooks have still to demonstrate their right to be styled 'adequate' anchors." In the first place, it should be remembered that slip is a necessary part of the anchorage. If there is no slip, the anchor is not loaded. The amount of the load carried by the anchor in a beam is indeterminate, but will be greater if the slip of the anchor is small. For this reason anchors may be advantageously compared by means of their loadslip curves, as obtained in this investigation. Just what constitutes a "small" slip depends upon the proportions of the beam. Tests on beams in which anchors have been used have shown substantial increase in carrying capacity, although the present investigation disclosed that the hooks used must have slipped quite appreciably. Tests by Bach show an increase in carrying capacity of about 50 per cent from anchors consisting of hooks of size about 8d. These hooks split the concrete at failure. The present investigation shows that the large hooks of diameter 12d have much better load-slip curves than the smaller hooks, carry a higher load, and are not likely to split the concrete. This would certainly seem to give promise of valuable gains from the use of these larger hooks.

T. D. MYLREA.—The German tests show an increase of approximately 50 per cent in the carrying capacity of beams when semi-circular hooks are Mr. Mylrea.

used (invariably followed by splitting), whereas in the beams I have shown the increase in the carrying capacity was more than 100 per cent, and there was absolutely no sign of splitting. I am inclined to think there are much better anchors than semi-circular hooks.

Mr. Slater. W. A. SLATER.—Suppose you were to use spirals around the ends of the hooks. I am wondering if the spirals in order to be close enough together to be effective would preclude getting sufficient concrete within the spiral.

Mr. Mylrea. T. D. MYLREA.—I think that was in part the difficulty with the specimens tested. We tried to get over it by screwing the spirals into the wet concrete. I imagine the difficulty would be increased when you tried to pour the concrete around and through the spirals.

Some Features of the Testing of Stevenson Creek Arch Dam.*

By Willis A. Slater.†

In the fall of 1925 the Bureau of Standards was called upon by Engineering Foundation for advice relative to the construction and for assistance in the testing of a concrete arch dam in California which was to be built and tested by Engineering Foundation Committee on Arch Dam Investigation. The dam was to be constructed for investigational purposes only without giving consideration to any possible value of the dam for service after completion of the tests. In fact it was to be tested to failure if this proved to be feasible and necessary. The writer was detailed by the Bureau of Standards to direct the tests for the committee in charge of the project. He spent the time from December, 1925, to September, 1927, in California on this work. Up to the time of completion of the load testing in September, 1926, most of the time was spent at the site of the dam. The remainder of the time was spent at Los Angeles working up a report of the construction and testing. The report, which has been published in the *Proceedings* of the American Society of Civil Engineers for May, 1928, covers in considerable detail the methods of construction and testing and the results of the test. The paper here presented gives principally methods used in the testing with only enough of the results to illustrate the kind of data secured, and the methods used in their interpretation. The data secured were voluminous and the reduction and interpretation occupied the equivalent of the full time of seven men including the writer for about ten months. Although many field tests of structures had been made during the previous 15 years and methods had been developed for systematizing the work, this test presented problems with regard to scope and required accuracy which were beyond similar problems in any other test with which the writer is familiar. Considerable time was consumed in finding a solution for the problems and developing and systematizing methods of interpretation, which would not be necessary in a repetition of a similar investigation.

The successful completion of the work would not have been possible except for the wholehearted co-operation of the crew of men who took the data and assisted in their reduction and interpretation. These men were G. H. Barry, R. W. Carlson, A. W. De Yoe, K. M. Fenwick, Inge

*Publication approved by the Director of the Bureau of Standards.
†National Bureau of Standards.

Lyse, L. J. Marchand, J. J. Stout and B. E. Wilson. Especial mention is made of the constructive part taken by Messrs. Carlson and Lyse.

The Stevenson Creek concrete arch dam is located about 60 miles northeast of Fresno, California. It is 60 ft. high, 2 ft. thick throughout the upper 30 ft. of its height and is 7.5 ft. thick at the bottom. Its

FIG. 1.—TEST DAM AND SURROUNDINGS.

upstream face is vertical and forms a portion of a circular cylinder having a radius of 101 ft. There is no reinforcement in the dam. The gorge into which the dam fits is approximately V-shaped, so that although the dam is 140 ft. long at the top it has no length at the bottom. The foundation and abutments are of solid granite. An undersluice 4 x 6.5 ft.

in minimum cross section was built under the bottom of the dam approximately on the center line in order to provide sufficient capacity to carry away the heaviest flow whenever it was desired to keep the reservoir empty. However, a heavy storm in the winter of 1926-27 caused the undersluice to clog, and the reservoir has been full of debris and water most of the time since then. A 24-in. and a 6-in. gate valve were installed for controlling the height of water during testing. Work on the excavation of the foundation for the dam began in August, 1925, and continued through the succeeding winter. Considerable damage and delay were caused by rains which washed down into the site of the dam, tunnel muck which had been deposited on the sides of the canyon during the construction of a water tunnel several years earlier by the Southern California Edison Company. Large rocks were thus loosened which, rolling

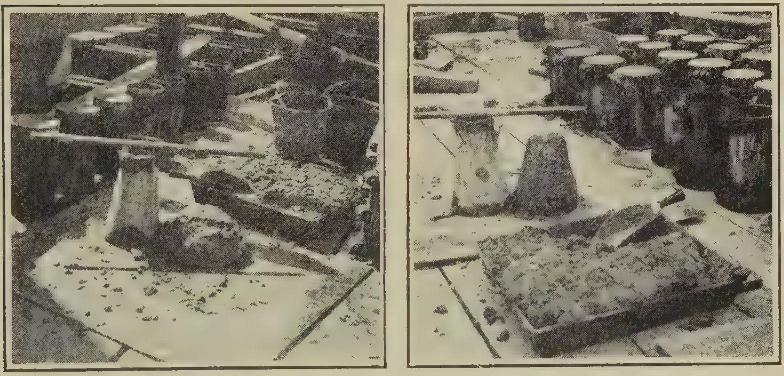

FIG. 2.—ILLUSTRATION OF CONSISTENCY OF CONCRETE.

down the canyon into the site of the dam did much damage. To overcome this the entire dump of tunnel muck (about 20,000 cubic yards) was washed out by hydraulic methods.

Placing of the concrete began on April 19 and was completed on June 4, 1926. The concrete was poured in lifts of 15 in., four lifts on each day of concreting, that is, about 5 ft. during the day. After each day of concreting about three or four days were required for building up the forms for another 5 ft. and placing the instruments for the next concreting. This gave construction joints every 5 ft. in height and a difference of about four days in the age of the concrete above the joint from that below the joint. The completed dam and the surroundings are show in Fig. 1.

Crushed granite was used for all aggregates. The largest size passed through a 2-in. round hole. The sand was that portion which passed a ⅜-in. square mesh screen. The mix stated in terms of these sizes was about

1:3:2 by loose volume. The range in consistency of the concrete is well illustrated in Fig. 2, which shows slumps of 2 and 6 in. The strength of concrete aimed at was 1800 lb. per sq. in. at 28 days. The strengths actually secured are shown in Fig. 3. The low strength near the bottom can be partly accounted for by the fact that a mix leaner than 1:3:2 was used in the first day's pouring. The strength variations near the top may have been due to the use of a new shipment of cement when the construction was nearly finished. The parallelism of the strength graphs for the specimens stored in damp sand and those stored at the dam, also for the specimens of different ages is good indication that the variations shown are largely actual variations in strength and not merely vagaries of testing.

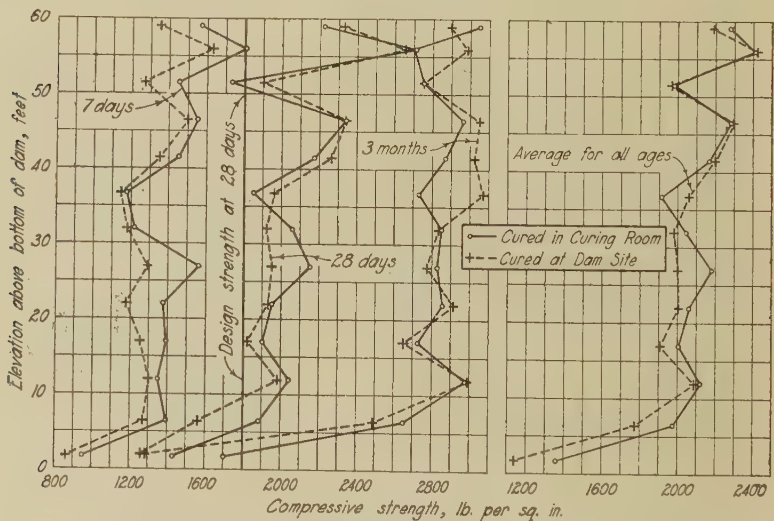

FIG. 3.—STRENGTH OF CONCRETE FROM VARIOUS ELEVATIONS.

For the lower 30 ft. of the dam the forms for the concrete were wetted with a hose three times a day after the placing of the concrete. This wetting did not seem adequate and from there to the top, water was kept flowing over the surface at all times that it did not interfere with the readings or other work. For this purpose a perforated pipe on the upstream face and another on the downstream face were kept at the level of the top of the concrete. These pipes were raised as the concreting progressed. After completion of construction a single pipe on the top of the dam delivered water for both faces of the dam. The location of this pipe is shown in Fig. 28.

All load tests were made at night in order to secure conditions of temperature stability and of uniformity of temperature throughout the dam.

The necessity for such a procedure is shown by the fact that on June 24, 1926, from 6.00 a. m. to 2.00 p. m., there were temperature changes which resulted in strains corresponding to stresses of approximately 100 lb. per

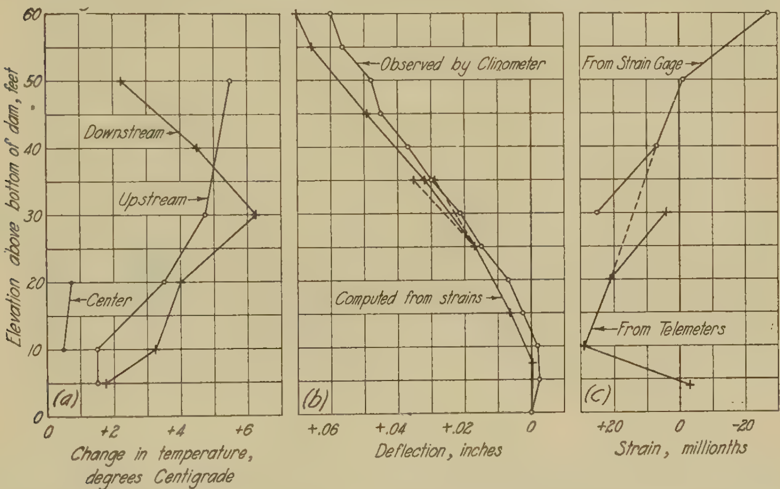

FIG. 4.—CHANGE IN TEMPERATURE, DEFLECTION, AND STRAIN ON JUNE 24 FROM 6.00 A. M. TO 2.00 P. M.

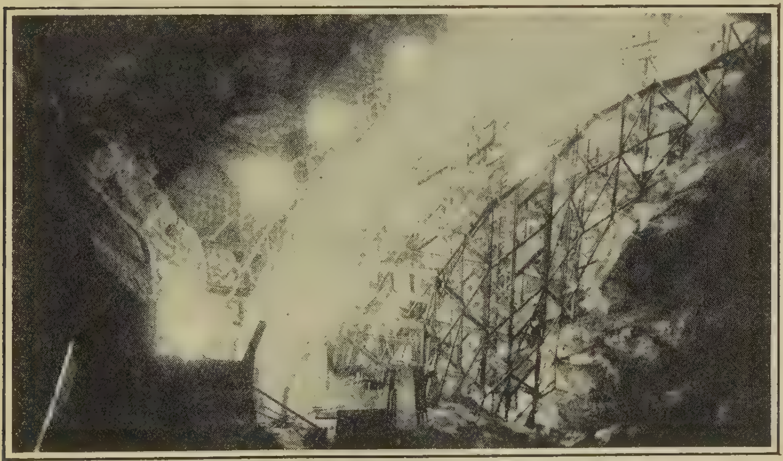

FIG. 5.—VIEW OF DAM TAKEN DURING TEST AT NIGHT.

sq. in. (See Fig. 4.) The fact that the greatest strains did not always occur at the positions where the temperature changes were greatest indicates the complicated nature of the behavior of the dam under tempera-

ture changes. For these night tests the dam and the observation platforms were well illuminated by electric lights as shown in Fig. 5.

In the testing, measurements were made of tensile and compressive strains, deflections, temperatures, and movements of abutments and foundation. Four different instruments were used for either the direct or indirect measurement of strains, (1) the strain gage, with which the surface strains on the downstream face and above water level on the upstream face were measured, (2) the electric telemeter, with which the strains within the concrete were measured, (3) the radius meter with which the bending strains on the downstream face were measured, and

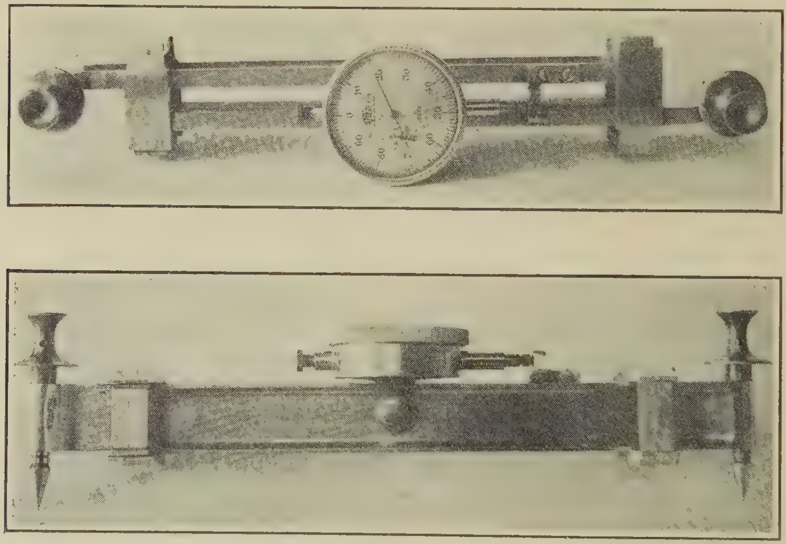

FIG. 6.—TYPE OF STRAIN GAGE USED.

(4) the clinometer, which measured the deflections accurately enough to permit determination of bending strains by differentiation of the deflection curves.

Fig. 6 is a photograph of the strain gage used. The principle upon which it operates is that of a parallelogram with one leg of the instrument attached to one corner of the parallelogram, and the other leg attached to the diagonally opposite corner. The two long sides of the parallelogram are connected by leaf springs. These springs are flexible enough to permit the necessary amount of movement and to serve as hinges between two sides of the parallelogram. The idea of this gage originated with H. L. Whittemore, of the Bureau of Standards who, at the request of the committee, designed and constructed one for its use. The final design of the instrument shown in Fig. 6 was worked up by

G. S. Binckley, of Los Angeles. The gages used were made of invar steel, having a coefficient of expansion of about 0.0000021 per deg. C., and were so constructed as to compensate almost completely for the effect of temperature changes in the instrument. Frequent readings were taken, however, on portable standard reference bars to facilitate the detection of accidental errors. The gage length was 10 in. for all strain gage readings.

The electric telemeter was designed by Burton McCollum and O. S. Peters, of the Bureau of Standards. As stated by McCollum and Peters in Technologic Paper No. 247, of the Bureau of Standards, the electric telemeter "depends upon the well-known principle that if a stack of carbon plates is held under pressure, a change of pressure will be accompanied by a change of electrical resistance and also by a change of length of stack, etc." By proper mounting and by calibrating the change of length against the change of resistance the carbon stack may be used to measure strains in a structure. The stack of carbon discs was mounted in a cylindrical steel case, (cartridge) having end bearing plates in contact with the carbon plates on the inside, and with the concrete on the outside, so that the deformation in 6 in. of concrete was transmitted to the carbon plates. Temperature changes within the stack cause expansion or contraction of the discs and other complications which affect the contacts and the resistance measured. It is necessary, therefore, to know the temperature changes within the cartridges in order to determine the temperature corrections to be made. These temperatures were measured by a resistance coil of No. 40 enameled copper wire which was included in each cartridge. With the coil used a change in temperature of one deg. C. caused a change in resistance of 2 ohms. The measurement of electrical resistance in the stacks of carbon discs and in the resistance coils furnished the data for determining both the strains and the temperature wherever the cartridges were placed. In all, one hundred and forty telemeters were used and these were well distributed throughout the dam. Both the strain gage and the electric telemeter measured the total deformations over the gage lengths to which they applied. Wherever a telemeter was placed on the downstream face another was placed in a corresponding position on the upstream face and the algebraic differences of the strains on the two faces gave the sum of the bending strains on the two faces. With the strain gage it was not possible to take observations below water level on the upstream face and it was necessary to find independent means for determining bending strains below water level since there were not enough electric telemeters to give the bending strains at all places where this information was needed.

For these additional measurements of bending strain, use was made of the radius meter and of the deflections measured with the clinometer. The instrument here termed radius meter was used to measure the change in mid-ordinate of a 40-inch arc of a horizontal element of the dam. The fact that the change in mid-ordinate is inversely proportional to the

radius of curvature of the deflection curve and proportional to the bending moment on the section indicates where the instrument gets its name and how the results may be used in the test. The instrument used (illustrated in Fig. 7) consists of a wooden bar with contact points at either end and an Ames gage at the center for measuring the mid-ordinate. Each .001-in. deflection on this 40-in. span corresponds to a bending stress of about 100 lb. per sq. in. in the upper half of the dam where the thickness is 24 in. It was difficult to be sure of the deflection within plus or minus about .0005 in., hence the usefulness of the instrument was limited. It is probable, however, that an instrument of this type can be developed which will be reliable and accurate. If this can be

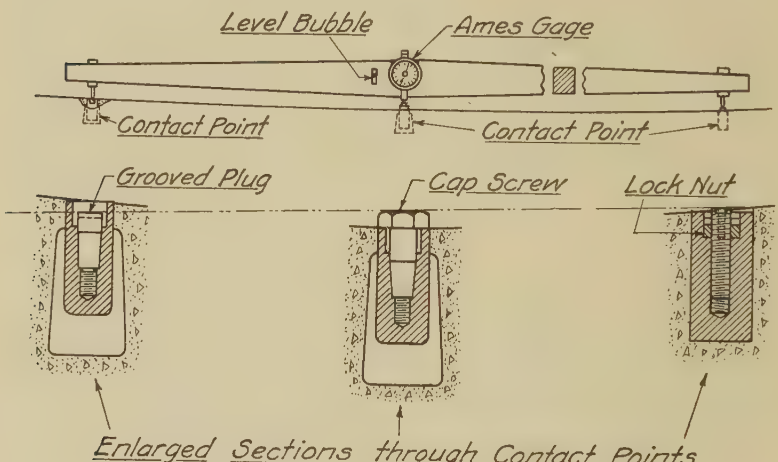

FIG. 7.—RADIUS METER AND DETAILS OF CONTACT POINTS.

done it appears to the writer that it will be a most useful instrument in structural research. In the test of the Yadkin River Bridge* an instrument of this type, designed by the Bureau of Public Roads, was used with results which speak well for its possibilities.

A second measure of the bending strains was afforded by differentiation of the deflection curves. The deflections were measured by means of a clinometer, which consisted of a vertical staff provided with, first, a level bubble used to bring the axis of the staff to a vertical position, second, a means of bringing the lower end of the staff to the same position relative to the dam each time it was used, and third, a micrometer at the top of the staff for measuring the variation in distance from a fixed point on the dam to the axis of the vertical staff. Some details of the clinometer and the brackets used in its application are given in Fig. 8.

*"Yadkin River Bridge Test Completed" *Public Roads*, Vol. 8, No. 10, p. 230, December, 1927.

Comparison of successive readings with the clinometer gave the horizontal deflection between successive stations which were 5 ft. apart vertically. The deflections so read are proportional to the changes in slope, hence one differentiation of the curve of measured deflections gives quantities proportional to M/EI, and one integration of the curve gives the deflection curve for the element to which the curve applies. The deflections were so determined on seven vertical elements, and it was therefore possible to determine deflection curves for horizontal elements as well as for vertical elements. Double differentiation of the deflection curves,

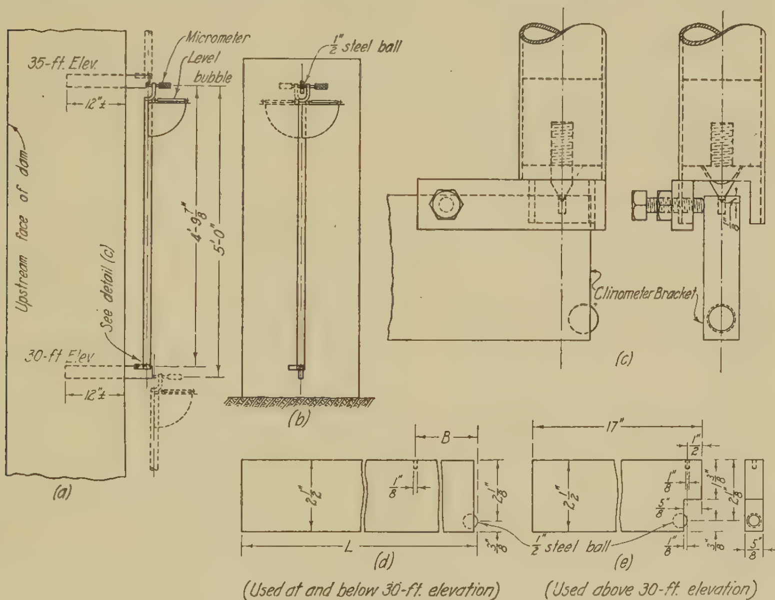

FIG. 8.—SKETCH OF CLINOMETER.

(a) AS USED FOR MEASURING DEFLECTION OF DAM.

(b) AS USED ON REFERENCE STANDARD.

therefore, permits the determination of M/EI and the strain for the horizontal elements also. The strains in the vertical direction were so determined at intervals of 5 ft. and were quite satisfactory. Those in the horizontal direction were determined at intervals of 10 and 20 ft., hence they were less satisfactory.

An instrument termed the level bar was used for measuring changes of inclination at the bottom of the dam. The principle of this instrument is the same as that of the clinometer, but it was used over a horizontal gage length of 10.3 in. instead of the 5 ft. used with the clinometer. Readings taken with it could be used in the same manner as

were those of the clinometer, but the results would probably be more subject to the effect of local conditions than are those of the clinometer, since it measures change of slope at a given point instead of the average change of slope in a distance of 5 ft. Employing, as it necessarily does, a shorter gage length than the clinometer, the percentage of error would be correspondingly greater. The slope of the brackets on which it operates is affected by temperature differences in different parts of the

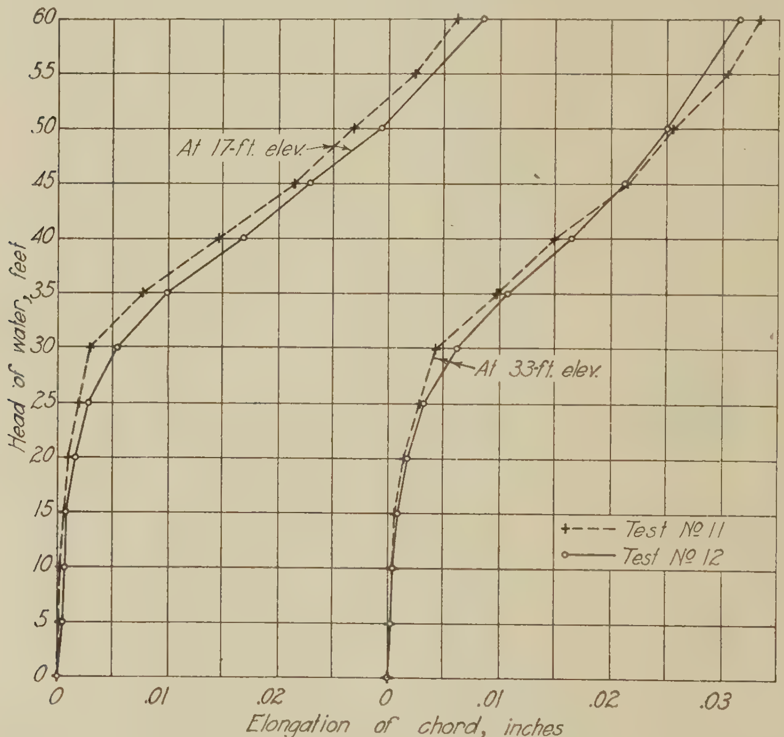

FIG. 9.—OBSERVED MOVEMENT OF ABUTMENTS AT ELEVATIONS 17 AND 33 FEET.

bracket, and this effect on the slope enters as an error in the results. With the clinometer only the change in length of the bracket affects the results and this change generally is negligible. For use with a horizontal instead of a vertical member the advantages and disadvantages of the two instruments would be completely reversed and the level bar would be more generally useful than the clinometer.

The movements of abutments were measured relative to each other at heights of 17 and 33 ft. A ⅜-in. dia. invar bar attached at one abutment extended along the direction of the long chord at each of these elevations to the opposite abutment where it bore against the plunger of an Ames

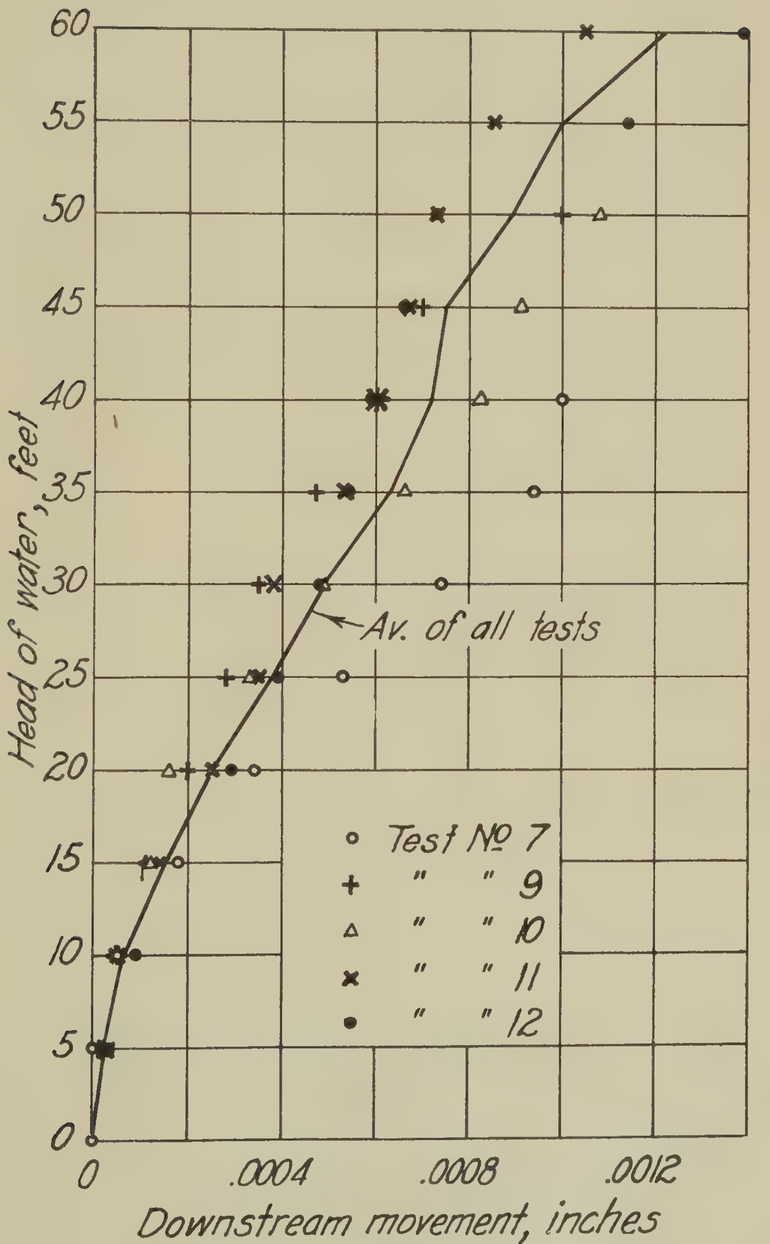

FIG. 10.—RADIAL MOVEMENT OF FOUNDATION ROCK AT ELEVATION 1 FOOT.

gage which was attached to the abutment rock. The readings were taken during the emptying of the reservoir. The use of invar for this bar reduced to a negligible quantity the change in length of the bar due to variation in temperature during the emptying of the reservoir, and the Ames gage measured the sum of the movements of the two abutments in the direction of the long chord. At the top of the dam it was not possible to provide an invar bar for the full length of the chord, and, instead, measurements were made of the total deformations in the abutment rock within a distance of about 10 feet at each end of the dam. The movement of abutments is assumed to be equal to the sum of these deformations at the two ends. Similarly to determine the downstream movement of the foundation of the dam at the bottom, the total deformation in the foundation rock in a distance of 11 feet downstream from the downstream face of the dam was measured, and this total deformation was taken as the total movement of the dam in a downstream direction. A ⅜-in. invar bar 11 ft. long was attached to the foundation

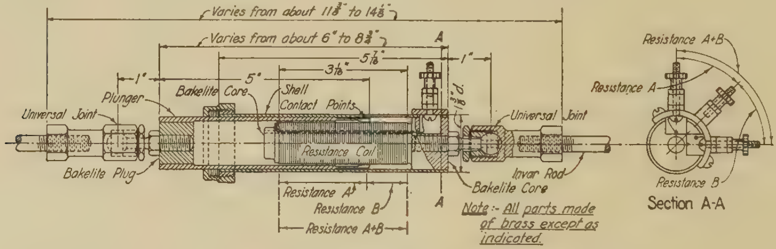

FIG. 11.—RESISTANCE MICROMETER..

rock at the downstream face of the dam at the bottom. The other end bore on the plunger of an Ames gage which was attached to the rock 11 ft. farther downstream.

The spreading of abutments at the 17 and 33-ft. elevations is shown in Fig. 9 for various heads of water in two successive tests. The measured downstream movement of the foundation of the dam at the bottom is shown in Fig. 10.

The opening of cracks above water was measured by means of a strain gage spanning the crack or by a metal bar attached to the dam close to one side of the crack and bearing on the plunger of an Ames gage attached to the dam on the opposite side of the crack. The opening of a crack under water on the upstream face between the dam and the foundation at the bottom was measured by means of a resistance micrometer developed for this test by G. S. Binckley, of Los Angeles.

The resistance micrometer is an instrument which consists essentially of a variable resistance with a sliding contact so arranged that the distance through which the contact moves is equal to the movement to be measured and approximately proportional to the change in resistance. The coil of resistance wire is wound around a core and each end of the

wire is connected with an instrument for measuring the resistance. A circular contact slides along the resistance coil, as shown in Fig. 11. A third wire is carried from this circular contact to the resistance-measuring instrument. As the deflection increases, the contact moves away from one end of the coil toward the other end, increasing one resistance and decreasing the other. Since the resistance of each part of the entire coil can be measured at any time, it is possible to determine the proportionate part of the entire range which has been traversed by the contact at any time. Although the resistance in the lead-wire may vary with the temperature, the proportional effect will be nearly the same on all three resistances so that the movements may be determined without correcting

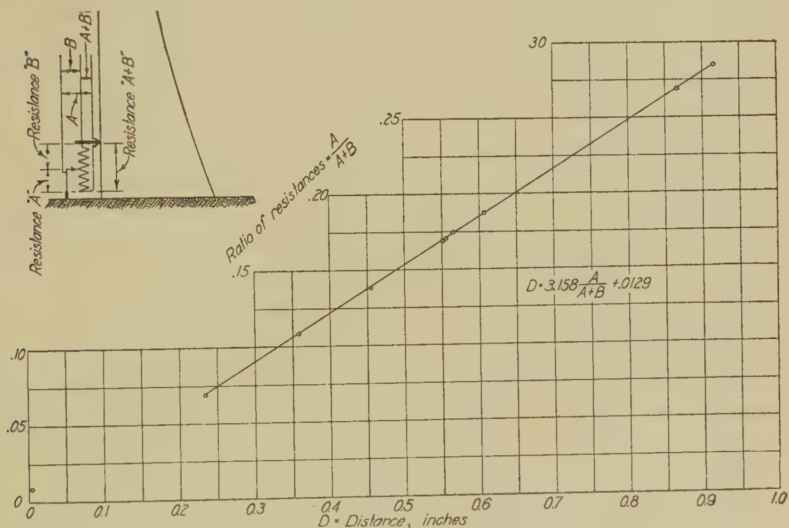

FIG. 12.—CALIBRATION CURVE OF RESISTANCE MICROMETER.

for temperature change, length of lead-wire, etc. The length of lead-wire will affect the proportionate resistances, but with wires of the size used in the test, this effect would be negligible.

A calibration curve for the instrument is shown in Fig. 12. This calibration was made just before attaching the instrument to the dam. No reference standard could be used during the test and dependence had to be placed upon the calibration curve. The instrument used was not designed for under-water measurements and in adapting it to the measurement in the change of crack width, the essential feature was its protection against leakage of water into the resistance coil and into the leads from the instrument to the Wheatstone bridge. To accomplish this the entire instrument was encased in a boot made from an automobile inner tube, and rubber-insulated wires were used. It operated satisfactorily for the

purpose of measuring the crack width. (See Fig. 13.) Since then a modified instrument has been built by Mr. Binckley, which is encased in a metal tube corrugated transversely (sylphon tube) to permit longitudinal movement under a small force and to exclude water.

The opening of a vertical crack, which formed under a head of water of about 47 ft. and extended from the top of the dam down to an elevation

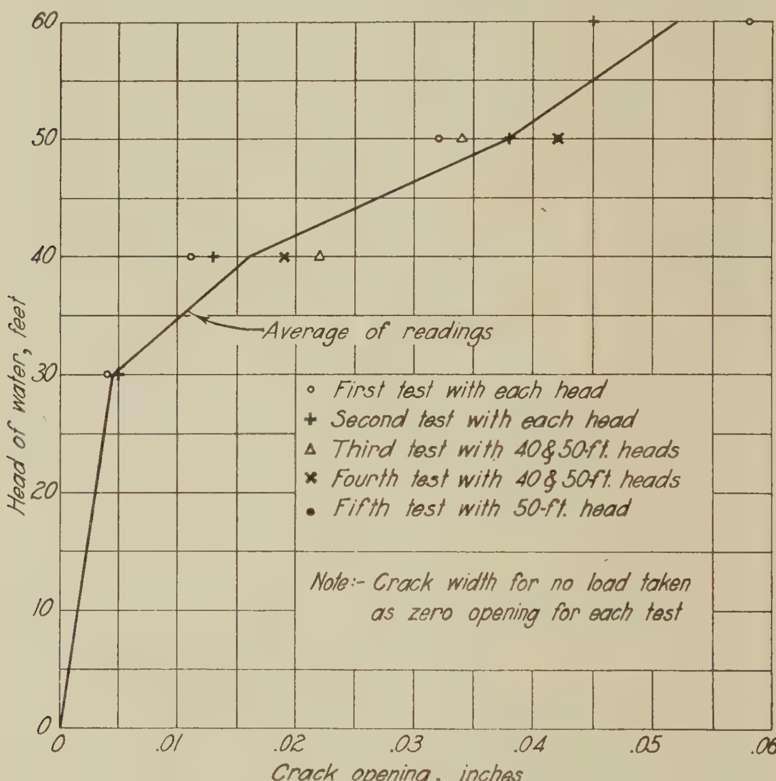

FIG. 13.—CHANGES IN WIDTH OF CRACK BETWEEN DAM AND BED ROCK ON UPSTREAM SIDE AT ELEVATION 2 FEET.

of about 49 ft. on the vertical center-line is shown in Fig. 14. The opening at the top of the dam was appreciable under a head of only 5 ft. and was a maximum under a head of between 45 and 50 ft. Its closing under the increased heads is apparently indicative of the influence of the increased arch stress at the top as the water rose. The locations of the stations for measurement with the strain gage, clinometer, telemeter, radius meter and level bar are shown in Figs. 15, 16 and 17. Fig. 15 also shows locations at which deflections relative to reference towers were to have

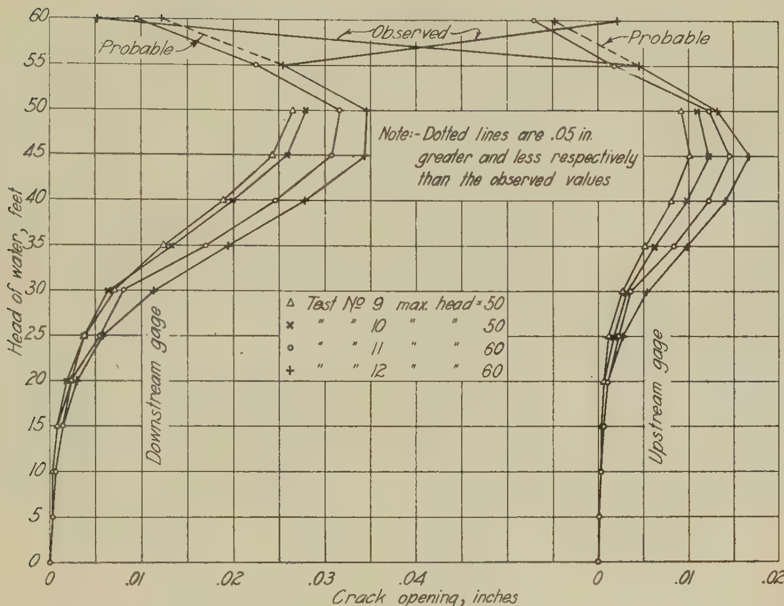

FIG. 14.—CHANGES IN WIDTH OF CRACK IN DAM AT CROWN, ELEVATION 60 FEET.

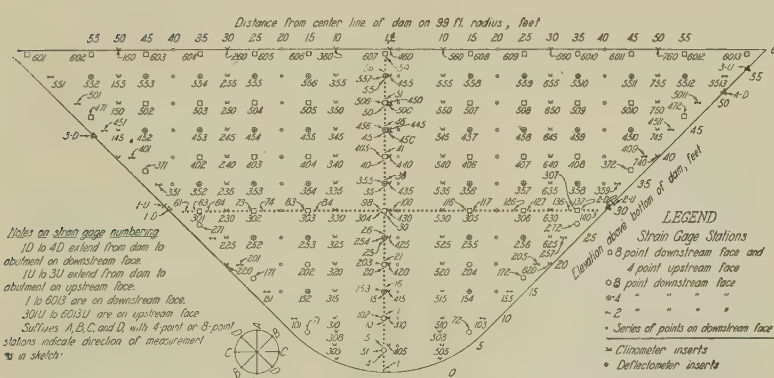

FIG. 15.—LOCATION OF STATIONS FOR OBSERVATIONS WITH CLINOMETER, STRAIN GAGE, AND DEFLECTOMETERS.

been measured by means of the resistance micrometer. These towers were not built and the deflections were not measured by this method. Fig. 17 shows also locations of wells in which thermometers were inserted for the measurement of temperatures. Fig. 18 shows the manner in which the telemeter cartridges, the clinometer brackets and the radius meter plugs

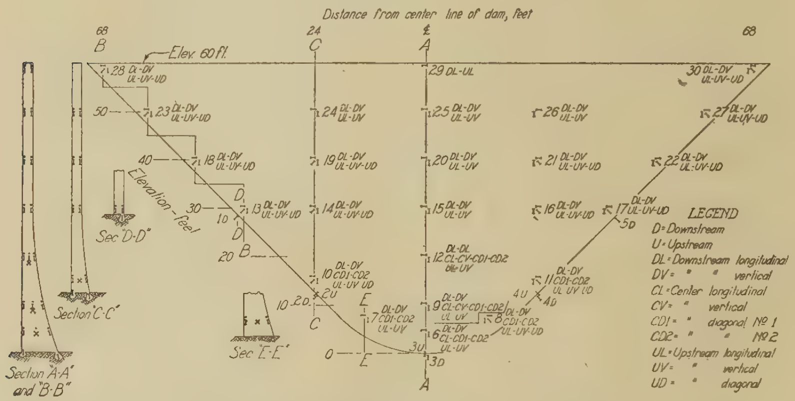

FIG. 16.—SKETCH SHOWING LOCATION OF TELEMETERS.

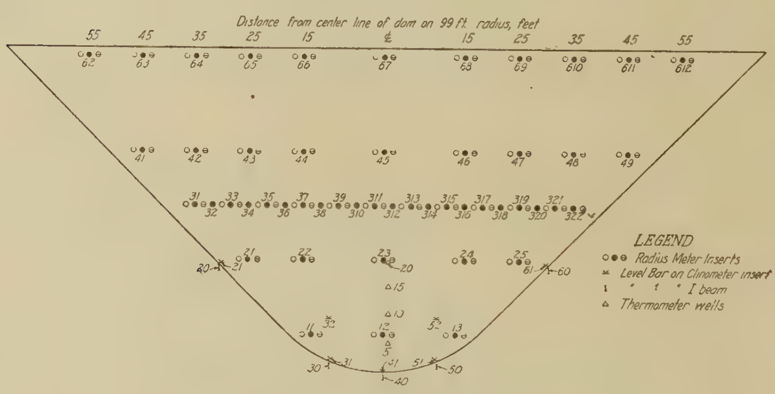

FIG. 17.—LOCATION OF STATIONS FOR RADIUS METER, LEVEL BAR, AND THERMOMETER.

were held in position during concreting. The telemeter cartridges near the upstream and downstream faces were held in place by means of wire brackets. Those for the center of the dam were cast into blocks of mortar which after hardening were supported in position by inserting into pipe sleeves properly placed in the concrete below, the projecting ends of short bars which had been cast into the mortar block. Metal sockets for the reception of the radius meter plugs and strain gage plugs were held in

FIG. 18.—CONSTRUCTION JOINT AT ELEVATION 20 PREPARED FOR NEXT POURING; TELEMETERS, CLINOMETER, BRACKET, AND RADIUS METER INSERTS IN PLACE.

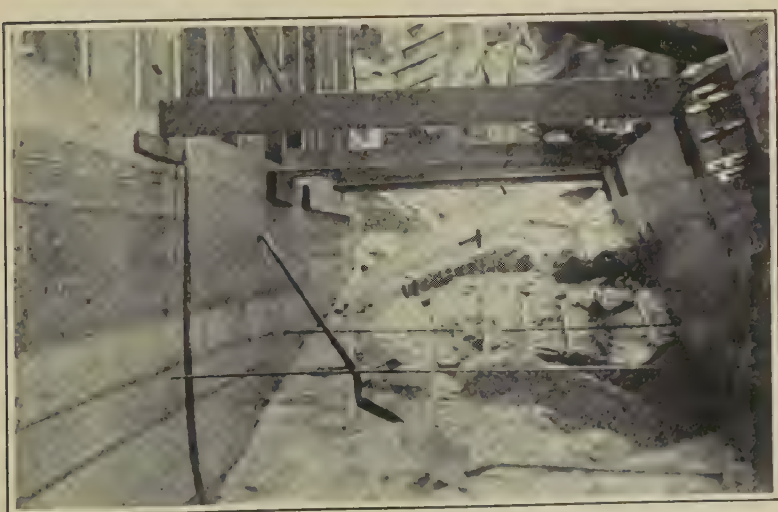

FIG. 19.—FOUNDATION AND INTERIOR OF FORMS BEFORE CONCRETING.

place during construction by means of thin metal templates attached to the forms. Templates for the radius meter plugs are shown in Fig. 18 and those for the strain gage plugs in Fig. 19. Fig. 19 also shows the condition of the foundation rock prior to concreting. The clinometer brackets are shown in detail in Fig. 8. The end of a bracket as it projected through the form to the interior of the dam is seen in Fig. 18. The exterior ends of the brackets projecting downstream are seen in Fig. 28.

Temperatures found in the interior of the dam (from the telemeter cartridges) at station 6, (elevation about 4 ft.) are shown in Fig. 20 from the date of pouring the concrete at this elevation (April 23, 1926) until the completion of the load tests on Sept. 22. These temperatures

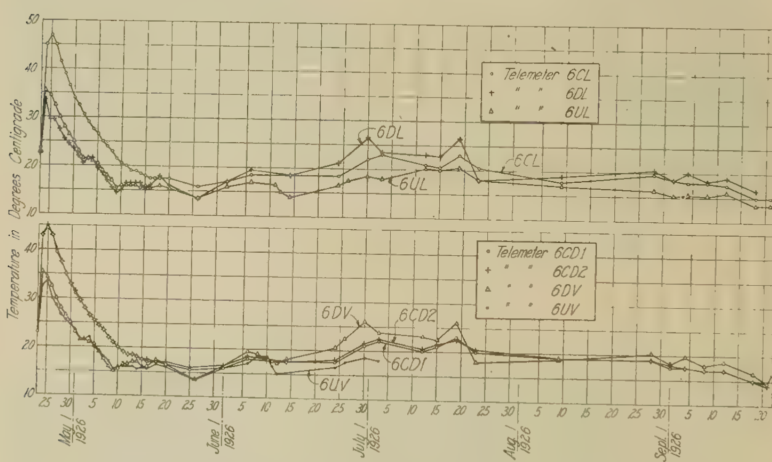

FIG. 20.—TEMPERATURES AT STATION 6 DURING CURING OF CONCRETE.

In the numbering, 6 stands for the station number, C stands for center, D for downstream, U for upstream; for the second letters, L stands for longitudinal (i. e., horizontal and parallel to the axis of the arcs), D for diagonal and V for vertical.

were determined by means of the temperature coils contained in the telemeter cartridges. A corresponding diagram for the strains measured by the telemeters (other than those due to loads) is shown in Fig. 21.

The deflections under the 60 ft. head are shown as broken lines in Fig. 22, plotted in isometric projection. In the same figure are shown equivalent deflections determined from a celluloid model of the Stevenson Creek Dam. This model was built and tested under the direction of Professor G. E. Beggs of Princeton University, and with the assistance of D. B. Sloan, a graduate student at Princeton. The model was an exact replica of the dam in proportions but was only one-fortieth of the size of the dam. It was loaded with mercury so that at corresponding points the pressure on the model was to that on the dam as 13.6 (specific gravity

of mercury) is to 40 (ratio of sizes). To obtain the equivalent deflections plotted in Fig. 22 those measured on the model were multiplied by a factor which takes into account the differences between the dam and the model with respect to size, modulus of elasticity and pressure per

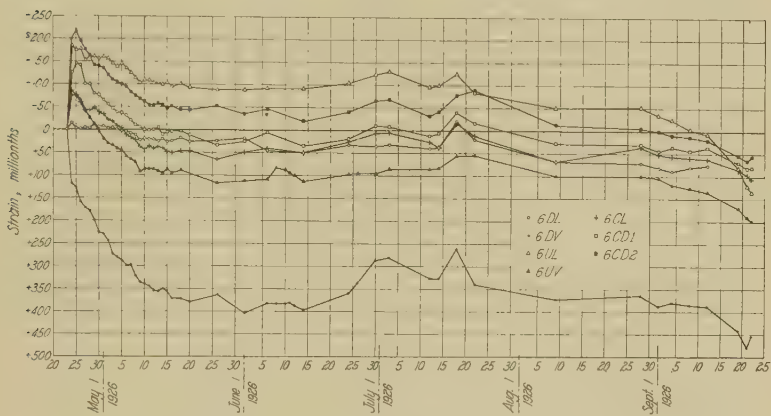

FIG. 21.—STRAINS AT TELEMETER STATION 6 WITH NO WATER IN THE RESERVOIR.

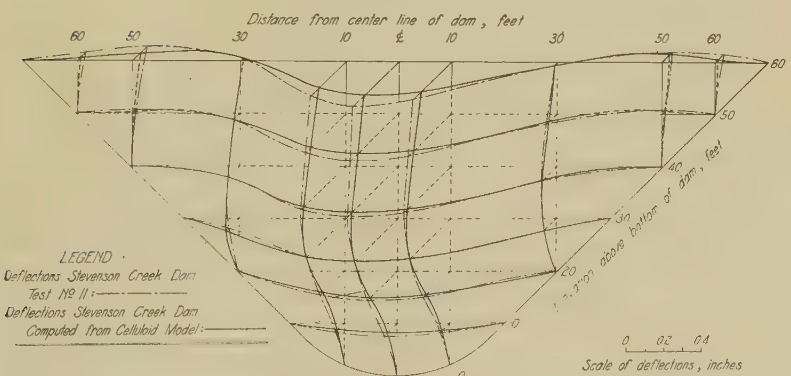

FIG. 22.—COMPARISON OF DEFLECTIONS OF MODEL WITH THOSE OF STEVENSON CREEK DAM UNDER 60 FT. HEAD OF WATER.

unit of area. The model was more nearly fixed in direction at the abutment line than was the dam and the difference shows up in the shape of the two sets of deflection curves. The agreement between the two sets of deflection curves is as good as can be expected in view of the difference between the restraint at the edges, and in view of the fact that there

were cracks on the vertical center line of the dam at the time of these measurements. The deflection curves give a good general idea of the manner of distribution of the bending stress and the results shown in Fig. 22 are suggestive of great possibilities in the use of models for the investigation of such problems as this.

The strain gage stations were laid out in such a way that at a large number of points on the downstream face of the dam strains were measured in four directions 45 deg. apart. This layout is shown in Fig. 15. From the strains measured in any three directions at a point the determination of the magnitude and direction of the maximum strain is possible. With strains measured in four directions four such determinations are possible. Fig. 23 shows the average magnitude and direction of the maximum strain so determined for each of the points for which the data were available. Below the 30-ft. elevation there was a tendency

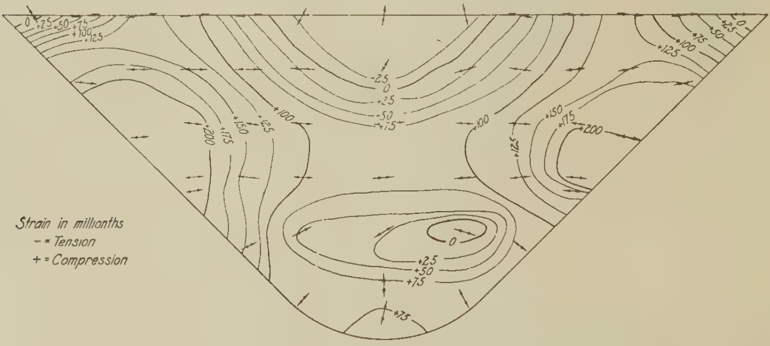

FIG. 23.—LINES OF EQUAL STRAIN AND DIRECTIONS OF PRINCIPAL STRAIN FOR 60 FT. HEAD OF WATER.

for the maximum strain at the abutment to approach a direction normal to the abutment line. Elsewhere it was generally horizontal but in a few places it was nearly vertical. If at a point in a vertical surface, one of the two principal strains is horizontal the other must be vertical. The indication, therefore, is that the common expedient of using horizontal and vertical co-ordinates for the computation of the stresses in a dam finds justification in the behavior of the dam as well as in expediency for the simplification of analysis.

On the vertical center line of the dam a continuous row of strain gage lines 10 in. center to center extended from top to bottom of the dam. Similarly at the 30-foot elevation a continuous row of strain gage lines 10 in. center to center extended the full horizontal length of the dam.

Fig. 24 illustrates the method by which the deflection and strain curves for the vertical sections were utilized. This figure shows results for the vertical center line, and the average for the two sections 10 ft. either side of the center line, and for the two sections 30 ft. either side

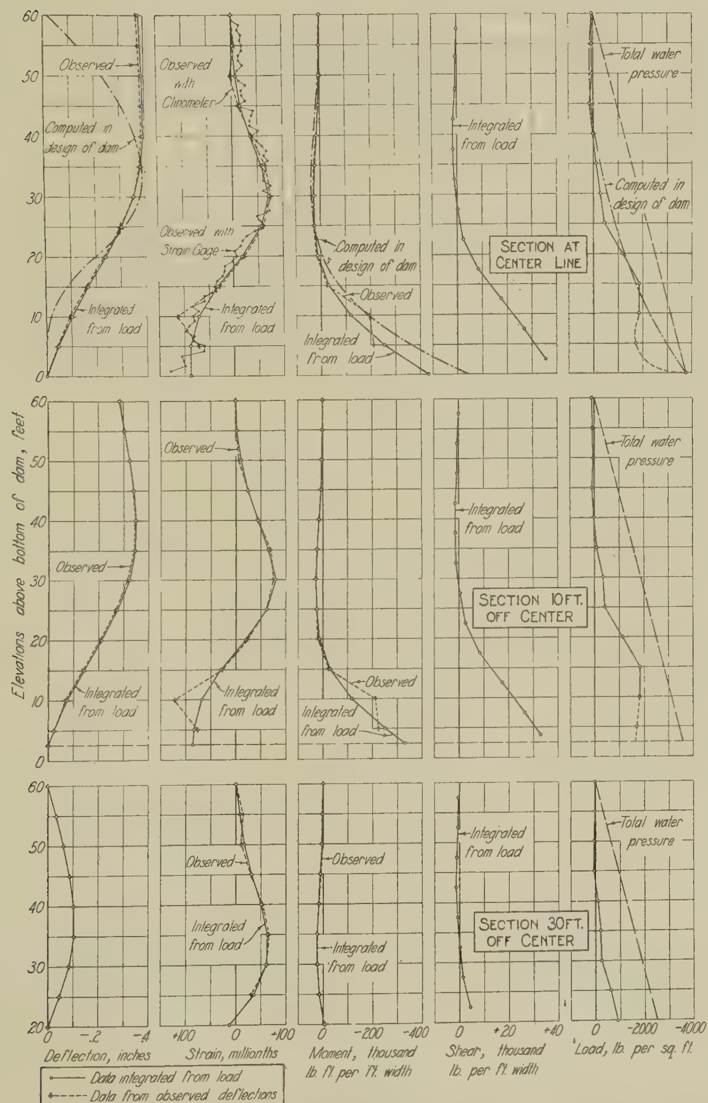

FIG. 24.—DEFLECTIONS, STRAINS, MOMENTS, SHEARS AND LOADS FOR VERTICAL ELEMENTS AT CENTER LINE AND 10 AND 30 FEET FROM CENTER LINE, HEAD OF WATER 60 FT., DAM CRACKED AT TOP AND BOTTOM.

of the center line. The graphs in the block at the extreme left show the deflections obtained by a single integration from bottom to top of the deflections measured with the clinometer. The graphs in the second block (toward the right) show the strains obtained by a single differentiation of the deflections measured with the clinometer. This is equivalent to a double differentiation of the deflections of the first block. As a

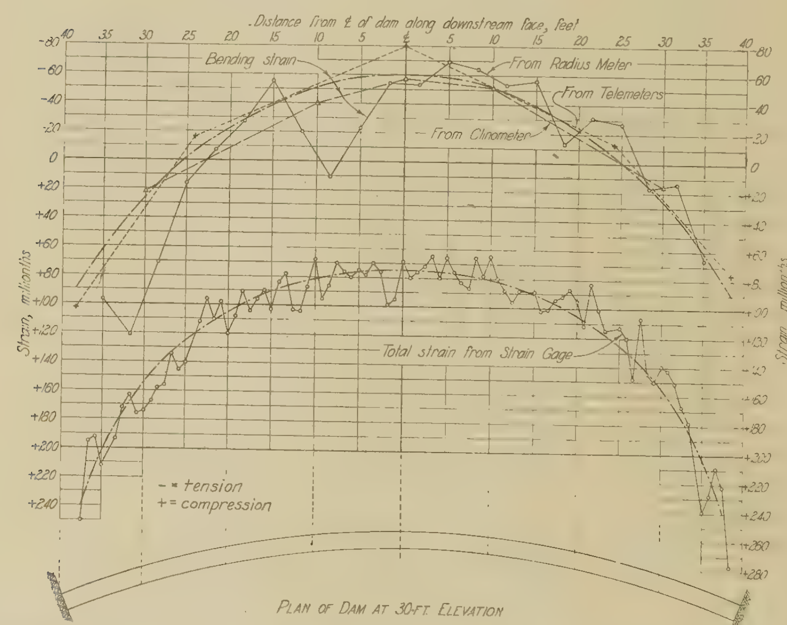

FIG. 25.—COMPARISONS OF BENDING STRAIN FROM CLINOMETER, TELEMETER, AND RADIUS METER DATA, WITH TOTAL STRAINS FROM STRAIN GAGE FOR HORIZONTAL ELEMENT AT ELEVATION OF 30 FEET UNDER HEAD OF 60 FEET.

check on the correctness of the strains obtained by differentiation of the deflections, the strains measured on the vertical center line with the strain gage are also shown in the second block at the top of Fig. 24. From the strains the bending moments, M, shown in the third block were computed from the equation

$$M = \frac{E\epsilon I}{t/2}$$

where E, ϵ, I and t are modulus of elasticity of concrete, measured strain, moment of inertia and thickness of the dam respectively. The shears in the fourth block were obtained by a single differentiation of the moment

curve and the loads in the fifth block by single differentiation of the shear curve. In so many differentiations there are great opportunities for error and repeated trial is frequently necessary to insure even approximate correctness in the differentiation. The test generally employed as a criterion for the correctness of differentiation was the integration of the differential curves for comparison with the curves from which they were derived. The moment and strain curves derived by successive integration of the load curves are shown in dotted lines and the agreement is seen to be very good. An additional check, available in some cases, is a comparison of the strains found from differentiation of the deflection curve with the strains measured with the strain gage. This has already been mentioned. The loads shown in the fifth block (the block toward the extreme right) are those which cause bending of the vertical elements. This load may be thought of as a load carried by bending instead of a load causing bending depending upon the point of view. Another portion of the load will be carried by direct thrust (arch action), and still another portion by bending in the horizontal elements. A further check is found in the comparison of the sum of the loads carried by the different means with the total load known to have been applied. This will be discussed later.

The total strains for the full length of the dam at the 30-ft. elevation, that is, the sum of the bending and the direct strains, as determined from the continuous row of strain gage lines, are shown in the lower graph of Fig. 25. The bending strains for the same elevation determined by three independent methods are shown in the upper group of graphs in the same figure. The difference between the total strain and the bending strain, represented in Fig. 25 by the vertical distance between the average graphs for the total and the bending strains, is the direct strain at the 30-ft. elevation, caused by the arch action as distinguished from the bending strain. The constancy of this direct strain throughout the length of the dam at this elevation, which may be noted in this figure, forms the experimental basis for the assumption of a constant arch thrust at all points at this elevation. These results are for a head of water of 60 ft. Similar results for other heads and for other elevations appeared to justify the use of this assumption as a working basis for all parts of the dam, though it is recognized that the arch thrust cannot be exactly equal at all points in any elevation.

The direct strains, ϵ, determined as shown in the preceding paragraph, have been used to compute the loads, p, carried by direct thrust (arch action) from the formula

$$p = \frac{E\epsilon t}{R}$$

in which R is the radius of the dam and the other symbols are as previously defined. Fig. 26 shows the strain, ϵ, and the loads, p, obtained for the 60-ft. head of water. At all stages of the test there was much

uncertainty regarding the conditions near the bottom of the dam. Under the 60-ft. head this uncertainty was greater than under the lower heads and is indicated by the dotted lines in the lower portions of the graphs. The increased uncertainty under the 60-ft. head probably is due mostly to the occurrence of a vertical crack near the bottom of the dam which was not there with the lower heads.

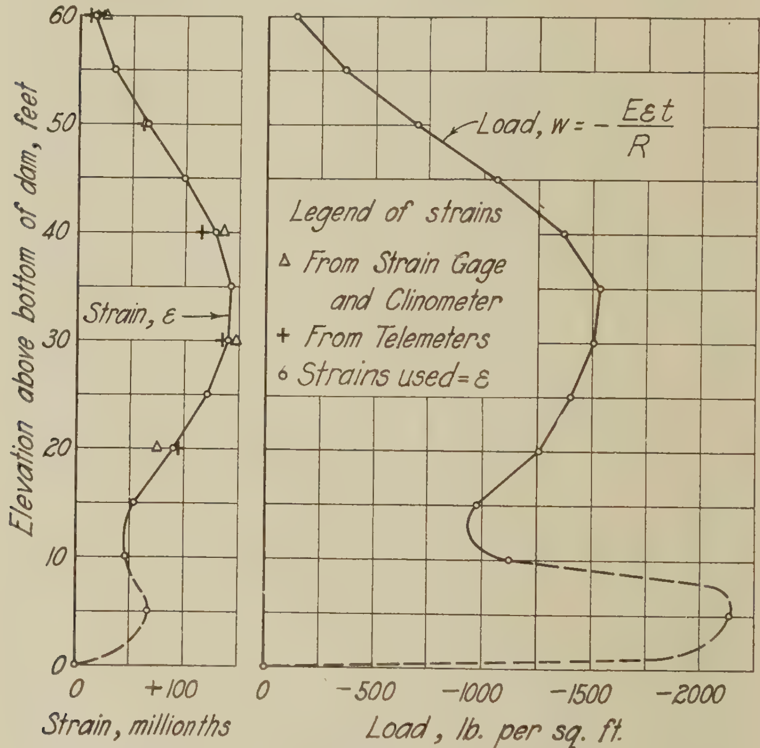

FIG. 26.—STRAIN AND LOAD DUE TO DIRECT COMPRESSION IN ARCH ELEMENTS FOR VARIOUS ELEVATIONS WITH WATER AT 60-FT. ELEVATION.

The loads carried by bending in the horizontal elements were found by successive differentiation of the curves of bending moment determined from the bending strains for those elements. As illustrated in Fig. 25, some of the curves for bending strains were quite irregular, and the obtaining of a reliable value for the load carried by horizontal bending was difficult. However, all the checks available were used and the results obtained are believed to represent the conditions fairly closely.

The previous discussion has treated the vertical and horizontal elements as though they were independent, that is, as though the sum

of the loads carried by direct thrust and by bending on any two of these elements at their intersection must equal the total water pressure within the area of the intersection. Evidently there will be a torsion in any two such intersecting elements, and this introduces a so-called equivalent

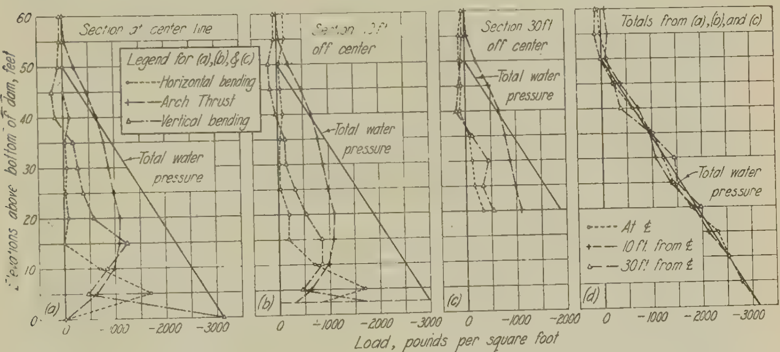

FIG. 27.—EQUIVALENT LOADS CARRIED BY HORIZONTAL BENDING, ARCH THRUST AND VERTICAL BENDING, COMPARED WITH THE WATER PRESSURE; HEAD OF WATER 60 FT.

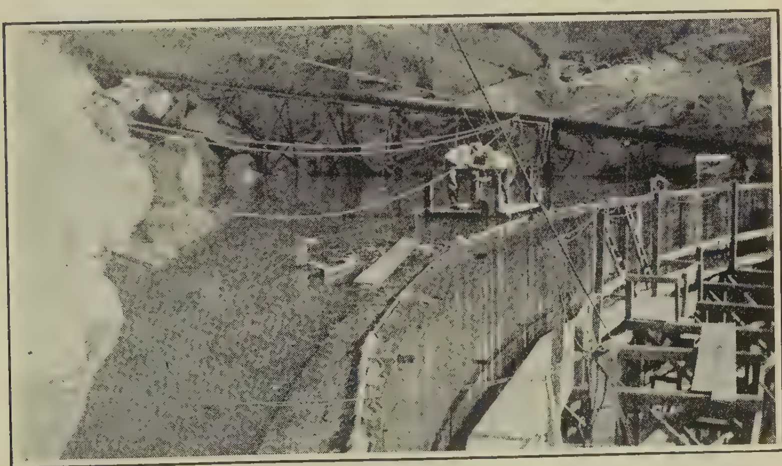

FIG. 28.—APPEARANCE OF DAM WITH RESERVOIR FULL.

torsional load which must be added to the direct and bending loads. These torsional loads were difficult to determine, but the data which were available indicated that they are so small everywhere in the dam as to be negligible. In the final checking up of the loads accounted for from the

FIG. 29.—APPEARANCE OF DAM AFTER FLOOD OF NOVEMBER, 1926.

test data, with the total pressure at a given elevation the torsional load has been neglected. Fig. 27 shows the loads found to have been carried by (1) horizontal bending, (2) arch thrust, and, (3) vertical bending at various elevations for three vertical sections of the dam, namely, the vertical center line section and sections 10 and 30 ft. from the center. In the block at the extreme right of this figure is shown a comparison of the total water pressure at various heights with the sums of the loads 1, 2 and 3 for the three sections mentioned above.

The appearance of the dam with the reservoir full is shown in Fig. 28. This figure shows also the lifeline provided for the safety of the men who operated the gate valves. The staging seen on the downstream side was used during the construction to support the form work, and

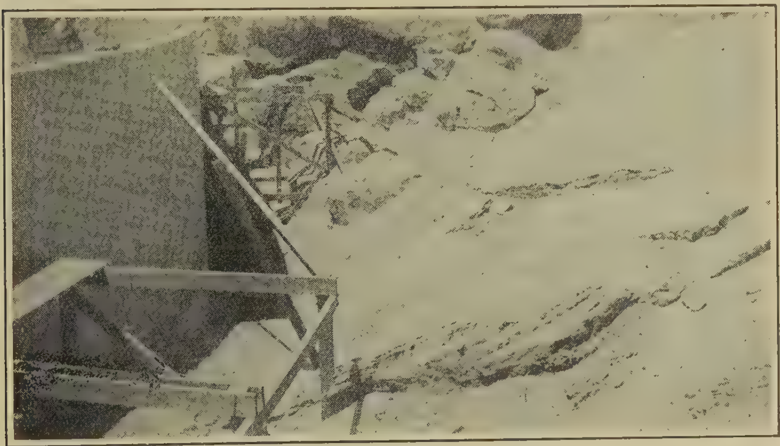

FIG. 30.—RESERVOIR FILLED WITH DEBRIS TO ELEVATION OF ABOUT 40 FT.

during the testing to support the observation platforms at elevations of approximately 15, 25, 35, 45 and 55 ft. Similar staging on the upstream side was submerged.

After completion of the load test the dam was left with the 4 by 6.5-ft. undersluice open but a storm in the latter part of November, 1926, choked the undersluice and destroyed the staging. Water flowed over the top of the dam to a maximum depth of about 3 ft. Fig. 29 shows the appearance after the severest part of the flood had passed. Some water flowed over the top of the dam until the summer of 1927. When the water had subsided it was found that the reservoir had been filled to an average elevation of more than 40 ft. with debris. Fig. 30 shows the appearance in the reservoir on July 14, 1927, after the water had subsided. The top layer of the debris consists of fine sand, but it is to be expected that below the surface large rocks would be found. On the

downstream face two cracks, one on either side of the vertical center line, have appeared since the completion of the load tests. Both cracks are about 20 ft. from the center line and they incline toward the center line as they extend upward. It is possible that the cracks were caused by impact of heavy rocks falling against the dam during the storm, but their symmetry with respect to the center line suggests that more likely the pressure of the debris has been enough greater than that of the water pressure to cause the cracks. The amount of pressure might be determined approximately by measuring, by means of the clinometer, the recovery in deflection of all parts of the dam on removal of the debris, and deducing from the deflections the load distribution. However, there has been so much opportunity for flow of the concrete under the continued load that the load so determined would not be likely to represent the highest pressure which at some time may have been applied by the debris. These are the only cracks known to have formed during the time of the storm and later. However, some of the other cracks are somewhat larger than they were previous to the storm.

In the upper left hand block of Fig. 24 there is shown for comparison with the measured deflections, on the vertical center line under a head of water of 60 ft., the deflections computed in the design of the dam. At mid-height the agreement is reasonably good. From the shapes of the curves it is apparent that in computing the deflections the dam was assumed to be fixed at the base whereas in the test it proved not to be entirely fixed. However, for the central vertical section the agreement of the moments and loads obtained from the tests with those used in the design was found to be reasonably good. The cylinder formula, which assumes the dam to be a portion of a complete circular cylinder (under external pressure) of the same height and the same radius as the height and radius of the dam was found to be inadequate for representing the conditions existing in an elastic arch of this type.

For the portions of the dam at and above the 40-ft. elevation the assumption of an elastic arch with uniform radial loading would not give stresses sufficiently close to those found from the test to warrant such an assumption as a basis for design. For the 30-ft. elevation the agreement of the stresses arrived at on the assumption of uniform radial load on the arch elements, with those found from the test was reasonably good. At this elevation the loads and stresses found from the test agreed with the results of analysis based on an assumed arch fixed at the ends, better than with an analysis based on an assumed hinged arch.

It seems evident from the test that no analysis of the dam which considers it merely as a series of disconnected arch ribs superimposed one upon another can give loads and stresses reasonably close to those actually resisted by the dam at all of its parts.

Of the instruments used in this test, the clinometer, the strain gage and the electric telemeter were about equally useful. The bending and direct strains could be determined by any two of them alone, but there

were certain places and functions for which each of the three instruments was particularly well adapted. These functions have been brought out in the test sufficiently that in a future test better application of their adaptabilities could be made. For example, telemeters and strain gages should be used on the upstream and the downstream faces of the dam, respectively, close to the abutments or other supports in order to determine both the bending and the direct strains at these positions. Although the bending strains were computed satisfactorily from the measured deflections, it was not possible to determine the strains in this way closer to the abutment or support than one full gage length away. For the vertical elements this was 5 ft.; for the horizontal elements it was much greater. With the telemeter and strain gage the strain may be determined at positions not over 5 in. from the support. This information would be of much value in determining load distribution at the bottom of the dam and at the abutments where so much trouble was experienced in this test.

The radius meter served the useful purpose of giving a check of the bending strains measured by other methods. This was especially important in the test of the Stevenson Creek Dam, which may be considered to be in the nature of a pioneer test, but it is an important consideration in any test.

In comparison with other field tests with which the writer is familiar, the test of the Stevenson Creek Dam has been highly successful. An important part of the success undoubtedly was due to the availability of instruments well adapted to the uses made of them, to the taking of readings at night, and to constant and untiring effort on the part of the observers. A feature, however, without which the test could not have been successful was the fact that it was possible to take the no-load and the load readings within so short a time of each other that there was not time for strains due to "flow" of the concrete, or other inelastic strains, to obscure the strains due to stress. Field observations on a number of service dams have been made by the Committee on Arch Dam Investigation. Some results of value have been obtained, but little information has been gained which throws light on the strains due to load, although the strain gage readings appear to have been taken carefully and accurately. The failure of these tests is due most likely to the fact that the long time elapsing between the no-load and the load readings permitted strain due to flow and shrinkage of concrete which were so large as to obscure the strains due to the stress. Tests are now being projected which include a study of methods of eliminating these disturbing features. Until such methods have been developed it seems that any field test should provide a method of taking no-load and load readings in close sequence if it is expected to determine strains due to loads.

Discussion.—Stevenson Creek Dam.

Mr. Davis.

R. E. Davis.—I think it would be of interest to those here to know something concerning the concrete that went into that dam. The aggregate was entirely granite containing a considerable amount of fine material—one test showed 26 per cent passing the 100 mesh sieve. One of the surprising things about the concrete was that this fine material apparently had little effect on the shrinkage which is contrary to what we usually find. We have been running a series of tests to determine permeability, and this concrete is unusually impervious. Under a 150-lb. pressure continued for 2 weeks on a disc 4 in. thick, the penetration was only 2 in. This is another example of what Professor White illustrated, that the swelling of the colloids keeps the water from penetrating farther into the mass. Another interesting fact we discovered is that a concrete in which the aggregate is granite has considerably less shrinkage or expansion, or I might properly say a less volumetric change with changes in moisture conditions, than does one where the aggregate is composed of gravel. Likewise the coefficient of thermal expansion for this particular concrete is lower than we ordinarily assume it to be for a concrete. All of which indicates that perhaps granite is not such a bad aggregate after all.

Flow of Concrete Under Sustained Compressive Stress.

By Raymond E. Davis.*

1. *Introduction.*—It is a fact long recognized that concrete is a material which, when subjected to sustained stress, continues to yield or to deform over a considerable period of time, so that the stress-strain relation accompanying the instantaneous application of load may be quite different from that existing after the lapse of time. The property of thus yielding, which has been referred to by various designations, will in this paper be termed *flow*, and it will be understood that flow includes only that plastic deformation which takes place subsequent to the application of load.

In order to learn more of the influence of various factors upon the flow of concrete the writer, in the summer of 1925, inaugurated a series of tests on plain concrete cylinders held in compression. These tests were designed to show the time-flow relation as affected by the following:

a. Cement ratio.
b. Gradation of aggregate.
c. Moisture conditions of storage.
d. Age at time of loading.
e. Magnitude of compressive stress.

This paper describes these tests and presents the results so far obtained.

2. *Acknowledgments.*—The tests described herein were conducted in the Materials Testing Laboratory of the University of California. The funds for carrying on the work were derived partly from grants of the Research Board of the University of California and partly by contribution of the Portland Cement Association to the Committee on Arch Dam Investigation of Engineering Foundation.

The experimental work was largely performed by Messrs. G. A. Sedgwick and K. B. Wolfskill, assistants in the Materials Testing Laboratory, acting under the supervision of Prof. G. E. Troxell.

3. *Scope of Tests.*—The tests are divided into two series which for convenience are designated after the year in which the work of observation was begun. In all, 69 specimens have been subjected to sustained load. In all cases the temperature has been maintained constant at 70 deg. F., and the relative humidity of the atmosphere surrounding the specimens stored in air has been 70 per cent.

1925 Series.—The specimens of this series were cylinders 6 in. in diameter by 24 in. long, stressed to 640 lb. per sq. in. To study the effect of

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* Professor of Civil Engineering, University of California, Berkeley, California.

richness of mix and of gradation of aggregate, the series was divided into four groups of six specimens each; one group of rich mix and high fineness modulus; a second of lean mix and high fineness modulus; a third of rich mix and low fineness modulus; and a fourth of lean mix and low fineness modulus.

To determine the influence of moisture conditions, one-half the specimens were stored in air of 70 per cent relative humidity after loading, and the remainder were stored in water spray at 70 deg. F.

In addition to the 24 specimens under load, four specimens, one from each group, were stored in air without being stressed, in order that corrections could be made in the observed deformations of the loaded specimens to allow for changes in length due to causes other than flow.

Also 6 x 12-in. cylinders of the same character of concrete as that used in making the 6 x 24-in. cylinders were made and were tested to determine

TABLE 1.—SIEVE ANALYSES OF AGGREGATES (1925 SERIES).

Sieve No.	Per Cent Passing	
	Niles Gravel	Niles Gravel and Rock
1 in.	....	83.3
¾ in.	....	59.5
⅜ in.	100.	38.5
4	96.1	25.4
8	66.3	15.0
14	40.8	9.1
28	23.5	5.9
48	9.2	2.4
100	2.0	1.0
Fineness Modulus	3 52	6.60

their ultimate compressive strength at the time the sustained loads were applied.

Santa Cruz portland cement was used, and the aggregate was Niles gravel for the mixes of low fineness modulus, and a mixture of Niles gravel and crushed rock from the quarries of the Oakland Building Materials Co. for the mixes of high fineness modulus. Table 1 shows sieve analyses of the two aggregates.

For the rich mixes the cement ratio was 1: 4; for the lean mixes the cement ratio was 1: 7. The relative consistency was 1.3.

After casting, the specimens were kept wet for 48 hours and then were immersed in water. Table 2 indicates the curing and storage conditions. This table also shows the nomenclature used in identifying the specimens, "D," standing for dry, being used to designate storage in air and "W," standing for wet, being used to designate storage in water spray; "R" referring to the rich mixes of 1 : 4 cement ratio, and "L" referring to the lean mixes of 1: 7 cement ratio; the number "1" standing for aggregate of high fineness modulus (6.60); and "2" for the aggregate of low fineness modulus (3.52).

Consulting the table, it will be seen that those specimens which were stored in air during the loading period were cured approximately two months in water and five months in air before the load was applied. The specimens which were kept wet after loading were stored continuously under water prior to loading. It will also be seen from the table that, in general, load was applied at the age of about seven months.

1926 Series.—The specimens of this series were cylinders 4 in. in diameter by 14 in. long, 45 being subjected to constant sustained loads producing compressive stresses from 200 to 1,200 lb. per sq. in. In addition to those specimens subjected to load, 15 additional cylinders of the same dimensions were stored without being stressed, in order that observations might be made and corrections applied to the observed deformations of the loaded specimens to allow for changes in length due to causes other than flow.

TABLE 2.—CURING CONDITIONS AND AGES OF SPECIMENS (1925 SERIES).

Specimen	Curing Conditions Prior to Loading		Age at Loading, months
	Months in Water	Months in Air	
DR1a	2.0	5.3	7.3
DR1b	2.0	5.0	7.0
DR1c	2.0	5.3	7.3
DR1d	2.0	5.2	Control
DR2a	1.8	5.4	7.2
DR2b	1.8	5.4	7.2
DR2c	1.8	5.4	7.2
DR2d	1.8	5.3	Control
DL1a	2.0	5.2	7.2
DL1b	2.0	5.2	7.2
DL1c	2.0	5.2	7.2
DL1d	2.0	5.3	Control
DL2a	1.7	5.3	7.0
DL2b	1.7	5.0	6.7
DL2c	1.7	5.4	7.1
DL2d	1.7	5.5	Control
WR1a	5.4	...	5.4
WR1b	7.4	...	7.4
WR2a	5.8	...	5.8
WR2b	7.1	...	7.1
WR2c	7.2	...	7.2
WL1a	7.3	...	7.3
WL2a	7.1	...	7.1
WL2b	7.2	...	7.2
WL2c	7.2	...	7.2

For all specimens, the concrete was of the same quality, being identical in character with that employed in the construction of Stevenson Creek Dam, upon which an exhaustive series of field tests has recently been completed under the direction of Engineering Foundation. It was composed of

1 part Colton cement, to 3.01 parts fine aggregate, to 1.15 parts medium-sized aggregate, to 0.89 parts coarse aggregate, measured by weight, the gradation of the aggregate being shown in Table 3. All of the aggregate, including the fine material, was granite.

The water-cement ratio was 1.03 by volume, or 0.64 by weight. The average slump, as determined by the standard slump test, was 3.2 in.

TABLE 3.—SIEVE ANALYSES OF AGGREGATES (1926 SERIES).

Sieve No.	Percentage Passing		
	Fine Aggregate	Medium Aggregate	Coarse Aggregate
1½ in.	....	....	86.7
1 in.	....	....	32.3
¾ in.	....	98.4	3.3
⅜ in.	100	26.5	....
4	85.3	2.1	....
8	67.3	1.4	....
14	50.3	....	....
28	33.4	....	....
48	17.3	....	....
100	9.0	....	....

TABLE 4.—CONDITIONS OF STORAGE AND LOADING (1926 SERIES).

Group	Curing Conditions	Storage While Loaded	Age at Time of Loading	Unit Stress lb. per sq. in.
1	2	3	4	5
WA2	Water	Water	2 days	200
WB3	"	"	7 "	300
WB6	"	"	7 "	600
WC3	Damp Sand	"	28 "	300
WC6	" "	"	28 "	600
WC9	" "	"	28 "	900
DC3	" "	Air at 70% humidity	28 "	300
DC6	" "	Air at 70% humidity	28 "	600
DC9	" "	Air at 70% humidity	28 "	900
WD6	" "	Water	3 mo.	600
WD9	" "	"	3 "	900
WD12	" "	"	3 "	1200
DD6	" "	Air at 70% humidity	3 "	600
DD9	" "	Air at 70% humidity	3 "	900
DD12	" "	Air at 70% humidity	3 "	1200

Table 4 indicates the curing conditions to which the specimens were subjected prior to loading, and the conditions which have been maintained subsequent to loading. The table also shows the age at which the load was applied and the intensity of compressive stress.

Consulting this table it will be seen that the series is divided into 15 groups of specimens some of which are stored in water at 70 deg. F.

and some of which are stored in air at 70 deg. F. and 70 per cent relative humidity.

In the nomenclature of the several groups shown in Column 1, "W" and "D" stand respectively for water and air storage. "A," "B," "C," and "D" stand respectively for the ages of 2 days, 7 days, 28 days, and 3 months, at which ages the loads were applied; and the numbers 2, 3, etc., indicate the compressive stress in hundreds of pounds per square inch. It is seen that 9 groups are stored in water, and 6 groups are stored in air, the loads of 200, 300, 600, 900 and 1,200 lb. per sq. in. being applied.

4. *Testing Apparatus and Methods.*—For the specimens of both series, deformations were measured with a strain gage, gage lines being at the third points of the circumference of each specimen.

For the 1925 series, deformations were observed with a Berry strain gage of 20-in. gage length; for the 1926 series, observations were taken

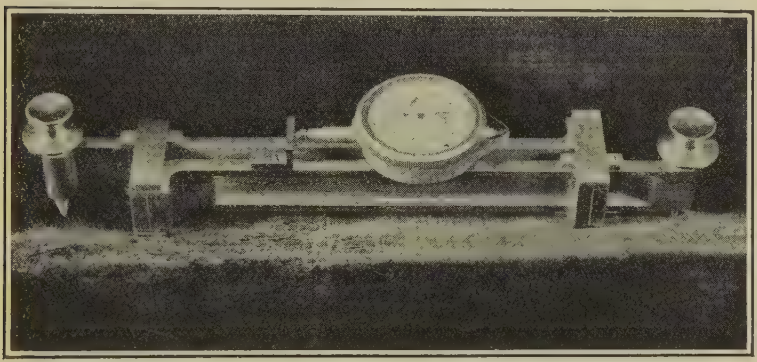


FIG. 1.—FULCRUM-PLATE STRAIN GAGE.

with a 10-in. fulcrum-plate strain gage based upon the principles patented by Whittemore but detailed and manufactured in the shops of the Materials Testing Laboratory of the University of California. Fig. 1 is a photograph showing the details of the fulcrum-plate strain gage. In passing it is perhaps appropriate to mention the marked superiority of this type of gage over any other so far devised.

In each specimen the load has been maintained constant by means of a car spring reacting against one end of the cylinder, car spring and specimen being clamped by a system of rods and plates, as illustrated in Fig. 2. The desired load is applied by compressing the car spring in a testing machine. In order to remove the disturbing influences of variable temperature the small portable testing machine shown in Fig. 3 was constructed specially for these tests, and during the process of loading was moved into the constant temperature room where the specimens were stored. Fig. 3 also shows a specimen in position in the testing machine

preparatory to loading. When the desired load, as indicated by the gage of the testing machine, had been applied, the nuts on the rods of the load-sustaining apparatus were brought to a bearing against the end plates. Thus, when the load on the testing machine was released, the compression in the spring was maintained by tension in the rods. This method of maintaining the load has proven very satisfactory, since it is possible to place the specimens in any position after the load has once been applied. Also the magnitude of the load may be readily determined at any time by placing the specimen in the testing machine, and observing the load at which the nuts again become loose. In no case has there been discovered

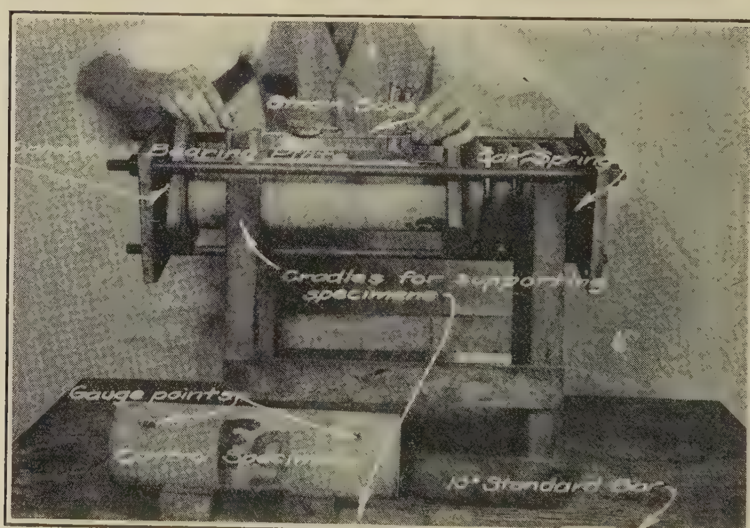


FIG. 2.—APPARATUS FOR MAINTAINING LOAD.

a measurable difference between the original load and that existing after an interval of time, even though that period be a year or more.

For those specimens stored in air a special insulated room was constructed, this room being equipped with electric heaters thermostatically controlled, and with special apparatus for maintaining constant humidity. Fig. 4 is a photograph showing some of the specimens in storage. By means of these controls it has been found possible to maintain temperature within 1 deg. F. and to maintain relative humidity within 1 per cent.

Fig. 5 shows a group of the specimens immersed in water, similar devices being installed to maintain water and surrounding air temperature within 1 deg. F.

The program of tests makes necessary strain-gage observations on each specimen just prior to and immediately following the application of

load, and at intervals thereafter, the time between observations being a day or less at first when the flow is rapid, and gradually increasing up to a month as the flow becomes less rapid. Accompanying these observa-

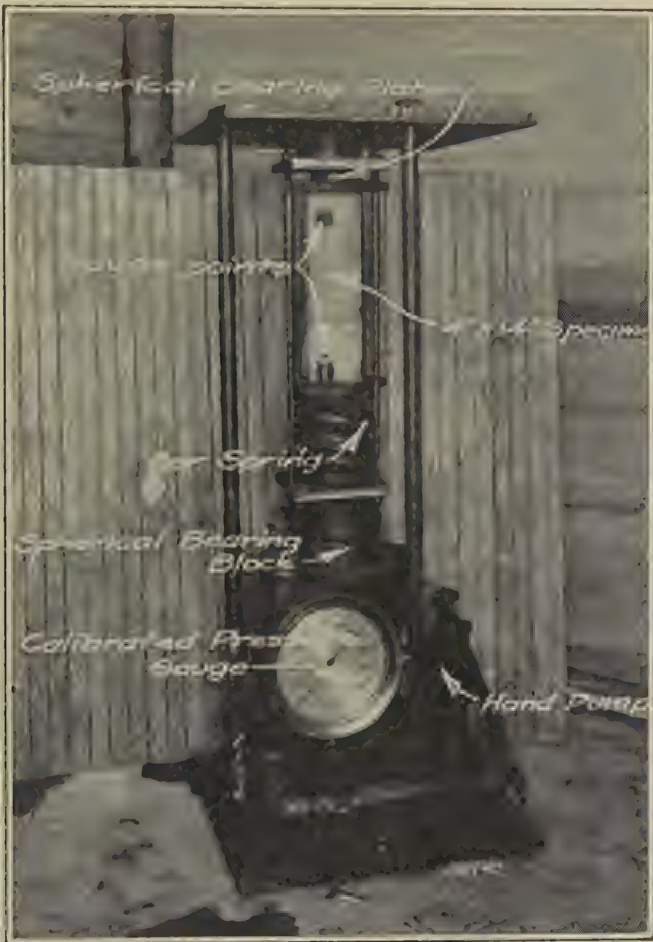


FIG. 3.—APPLYING LOAD TO SPECIMEN.

tions strain-gage measurements are, in general, taken on control specimens, in order that changes in length of the loaded specimens due to causes other than flow may be determined.

5. *Results of Tests (1925 Series).*—In Figs 6a, 6b and 7a, 7b are

shown diagrams for each of the several groups of this series, the magnitude of the flow being indicated by the ordinates and the time loaded in days being given by the abscissae. It is seen that the record extends over a period of about 150 days. The values given by the diagrams of Figs. 6a, 6b, which

FIG. 4.—STORAGE ROOM.

are for specimens stored in air and loaded at the age of about 7 months, are net flows, corrections for shrinkage having been made and the elastic deformation taking place upon the application of load having been deducted. The values given by the diagrams of Figs. 7a, 7b, which are for

specimens stored under water until loaded at the age of about 7 months and thereafter subjected to water sprays, represent the combined effect of flow and the expansion subsequent to loading due to whatever cause.

Consulting the diagrams of Figs. 6a, 6b, it is seen (1) that at the end of the 5-months' period over which observations extended the flow had in no case ceased; (2) that for the same character and gradation of aggregate the leaner the mix the greater the flow; and (3) that for the same cement ratio the lower the fineness modulus the greater the flow. For each group the rate of flow is greatly reduced after the first ten days.

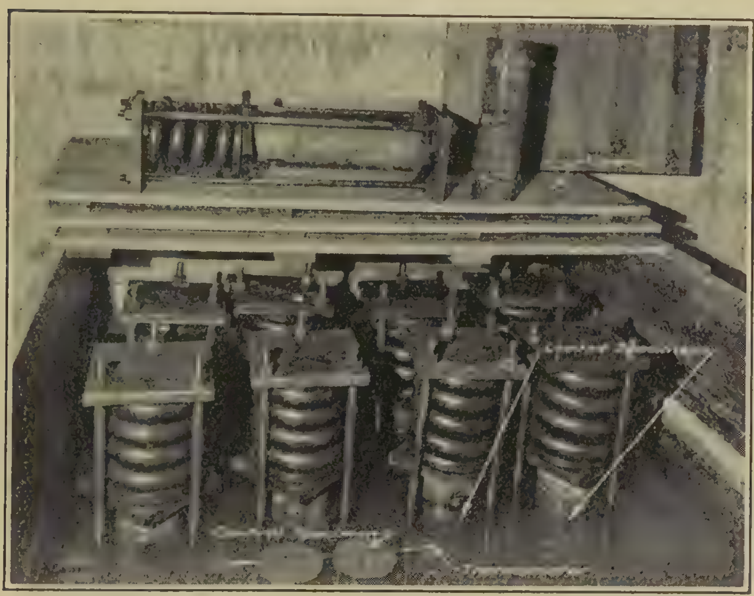


FIG. 5.—SPECIMENS IMMERSED IN WATER.

Except for the group of lean mix and low fineness modulus (DL2 group), it appears that the flow has nearly ceased at the termination of the 5-months' loading. The maximum flow of the DL2 group is more than ½ in. per 100 ft., but for the DR1 group (high fineness modulus and rich mix) the maximum flow is only about ¼ in. per 100 ft.

Consulting the curves of Figs. 7a, 7b, for which the specimens were wet continuously by spraying subsequent to loading, it is seen that during the first ten days or so the flow was rapid, but thereafter its rate decreased rather abruptly until at the end of 30 to 60 days its rate was equalized by swelling and the specimens began to elongate or to expand due to other influences than that of compressive stress. It appears that had the tests

been long continued this expansion after the cessation (or reduction of the rate) of flow would have perhaps exceeded the total flow and the net result would have been an elongation. So far as the influence of richness of mix and gradation of aggregate upon the flow are concerned, the dia-

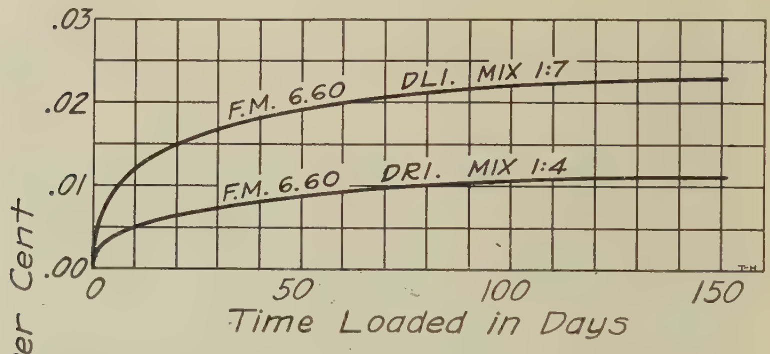


FIG. 6A.—FLOW OF SPECIMENS STORED IN AIR (1925 SERIES).

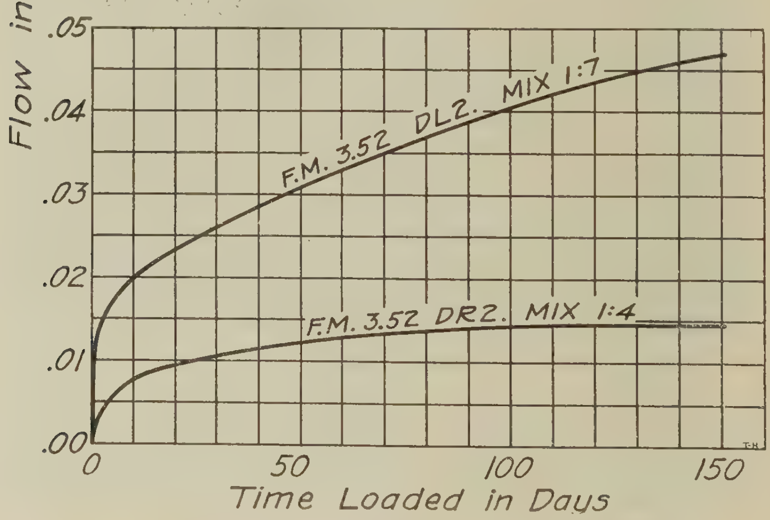


FIG. 6B.—FLOW OF SPECIMENS STORED IN AIR (1925 SERIES).

grams indicate the same general effect as for the specimens stored in air, that is, other things remaining equal, the leaner the mix and the lower the fineness modulus, the greater the flow.

Comparing Figs. 6a, 6b, and 7a, 7b, it is seen that the flow of the specimens stored in water spray is materially less than the flow of correspond-

ing specimens stored in air. Thus for the groups of lean mix and low fineness modulus, the maximum flow of the water sprayed group (group WL2, Fig. 7b) is 0.009 per cent, less than one-fifth the flow which had taken place in the corresponding group of specimens stored in air (DL2 group, Fig. 6b) at the end of the 5-months' loading period.

At the end of the 5-months' loading period the specimens were tested to their ultimate compressive strength, the stress-strain relation being determined. Table 5 shows for each group the ultimate compressive stress and the secant modulus of elasticity at 1,000 lb. per sq. in.

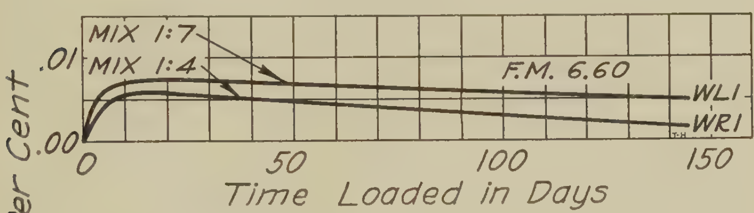


FIG. 7A.—FLOW OF SPECIMENS STORED IN WATER SPRAY (1925 SERIES)

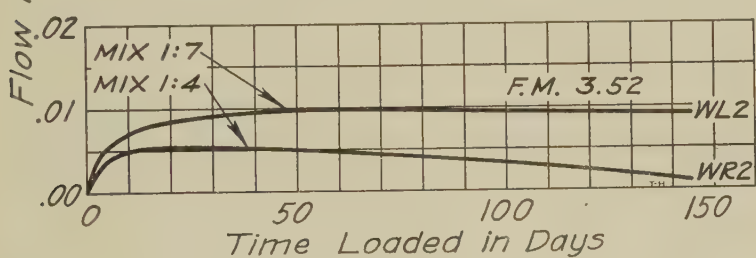


FIG. 7B.—FLOW OF SPECIMENS STORED IN WATER SPRAY (1925 SERIES).

It is interesting to note that, while without exception the ultimate strengths of the specimens stored in air are higher than for corresponding specimens stored in water spray, the reverse relation is, in general, true when the corresponding secant moduli are compared.

6. *Results of Tests (1926 Series).*—Figs. 8 to 11 give graphically the history of the several groups of the 1926 series, for which stress and age at time of loading were varied but the quality of concrete was the same for all specimens. For each of these figures, the ordinates of the curves are net flows in per cent, corrections having been made for volumetric changes due to other causes than stress, and the abscissae represent times in days from the instant of application of load. Near the right end of each of the figures there is also shown by vertical lines the corresponding deformations which took place immediately upon the application of load.

These have been termed the *elastic deformations.* Thus there is afforded a ready means of comparing flow taking place subsequent to loading with the elastic deformation produced at the instant of loading. In the figures the full-line curves are for groups stored in water and the dash-line curves are for those stored in air. It is seen that the period of sustained loading extends over about one year. Each plotted point represents the average flow of the several specimens in the given group at a given time.

Fig. 8 gives the time-flow curve for the specimens loaded at the age of two days, the sustained compressive stress being 200 lb. per sq. in., and the specimens being stored under water. It appears that after a year the flow, amounting to nearly 0.02 per cent, has practically ceased. This flow is somewhat more than half the elastic deformation.

Fig. 9 gives the results obtained with the two groups loaded at the age of seven days and stored under water, one group being stressed to 300 lb. per sq. in., and the other being stressed to 600 lb. per sq. in. It

TABLE 5.—COMPRESSIVE STRENGTH AND SECANT MODULUS OF ELASTICITY AT 1000 LB. PER SQ. IN. (1925 SERIES).

Group	Mix	Fineness Modulus	Ultimate Strength, lb. per sq. in.	Secant Modulus, lb. per sq. in.	Storage
DR1	1:4	6.60	4540	4,090,000	Air
DR2	1:4	3.52	3600	3,170,000	Air
DL1	1:7	6.60	2170	3,570,000	Air
DL2	1:7	3.52	2160	2,470,000	Air
WR1	1:4	6.60	3850	5,380,000	Water
WR2	1:4	3.52	3320	4,670,000	Water
WL1	1:7	6.60	1990	3,080,000	Water
WL2	1:7	3.52	1890	3,060,000	Water

appears that the flow taking place under a stress of 600 lb. per sq. in. is more than four times that taking place under a stress of 300 lb. per sq. in. At the end of a year the flow is a little less than the elastic deformation which took place at loading. At the lower stress the flow appears to have nearly ceased, but at the higher stress it is still increasing slowly, at the end of one year amounting to nearly ¾ in. per 100 ft. It is instructive to note that most of the flow took place within the first three weeks.

Fig. 10 gives the results obtained with the six groups loaded at the age of 28 days, three groups stressed respectively to 300, 600, and 900 lb. per sq. in. being stored under water, and three groups similarly stressed being stored in air. At the end of a year for each of the three stresses the flow of the specimens stored in air is roughly double that of corresponding specimens stored in water and is materially greater than the corresponding elastic deformation. Also it appears that for either wet or dry specimens the flow which takes place within the period of a year is proportionally greater for the higher stresses; that is, the flow at a stress of 600 lb. per sq. in. is more than twice as great as that at 300 lb.

per sq. in.; and the flow at 900 lb. per sq. in. is more than 3/2 as great as that at 600 lb. per sq. in. At the end of a year the water soaked specimens subjected to stresses of 300 and 600 lb. per sq. in. appear nearly to have reached a state of flow equilibrium; but all others are still flowing, and the air dry group stressed to 900 lb. per sq. in. seems to be decreasing in length rather rapidly. This latter group (DC9) exhibits a flow of more than 1 in. per 100 ft.; the corresponding water soaked group (WC9) shows a flow of more than ½ in. per 100 ft.

Fig. 11 gives the results obtained with the six groups loaded at the age of 3 months, conditions being similar to those described in the preceding paragraph except that stresses of 600, 900, and 1,200 lb. per sq. in., instead of 300, 600, and 900 lb. per sq. in., were employed. Examining the curves it is seen that at the end of a year, for each of the three

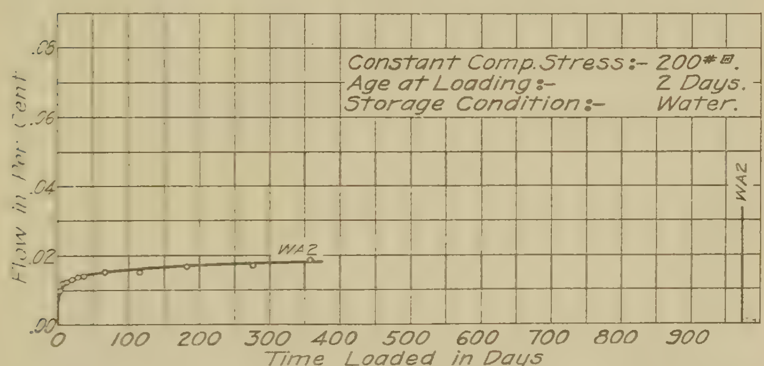


FIG. 8.—FLOW OF GROUP LOADED AT AGE OF TWO DAYS (1926 SERIES).

stresses, the flow of the specimens stored in air is about three times as large as those stored under water and is 1½ to 3 times as great as the corresponding elastic deformation. The rate of flow after one-year's loading is small for the specimens in water but is still relatively rapid for the specimens in air. The stress of 900 lb. per sq. in. sustained for a year produces flows of about ⅞ in. per 100 ft. for the group in air and less than ¼ in. per 100 ft. for the group under water. At 1,200 lb. per sq. in. the corresponding flows are 1¼ and ½ in. per 100 ft.

Comparing the curves of Figs. 9, 10 and 11 it will be found that the age at time of loading has a marked influence upon the magnitude of the flow taking place under a given compressive stress in a given interval of time. Thus after one year of sustained load in water at 600 lb. per sq. in. the flows in per cent are 0.057, 0.031 and 0.016 respectively for ages at the time of loading of seven days, 28 days, and three months. In Table 6 are given, for each of the various groups of specimens, the flow after a year of sustained load, the elastic deformation at the instant of loading,

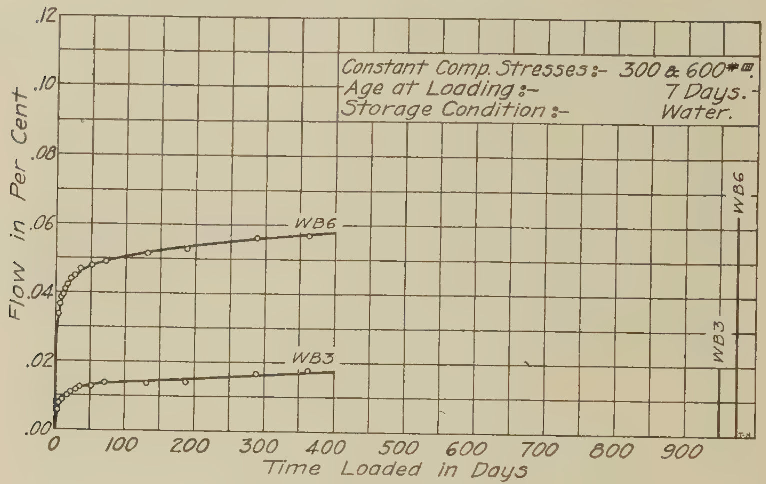


FIG. 9.—FLOWS OF GROUPS LOADED AT 7 DAYS (1926 SERIES).

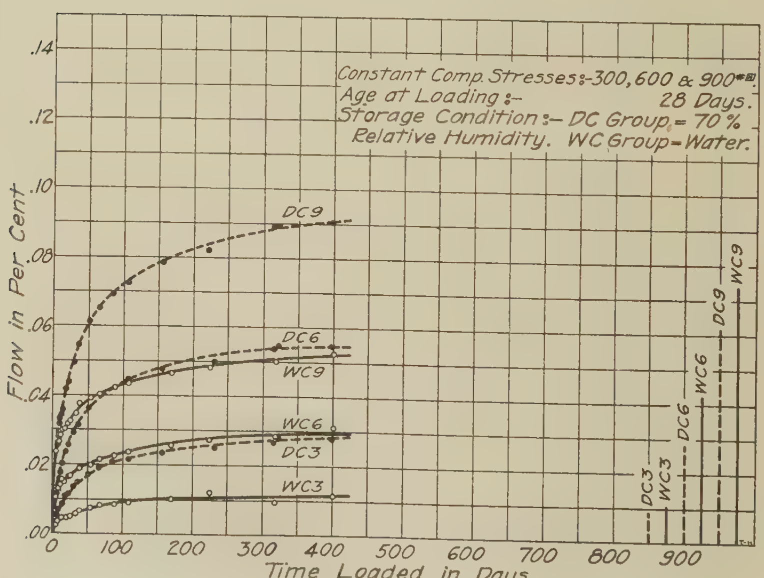


FIG. 10.—FLOWS OF GROUPS LOADED AT AGE OF 28 DAYS (1926 SERIES).

the sum of these two quantities, and the ratio of the flow to the elastic deformation.

7. *Effect of Time and Other Factors on Flow (1926 Series).*—In order to indicate the effect of length of time of sustained load upon the stress-strain relation, the stress-strain curves of Figs. 12 to 17 have been prepared. In each of these figures the ordinates are unit compressive stresses and the abscissa are total unit deformations due to stress, including the elastic deformation at the instant of application of load and the flow subsequent thereto. Each figure gives values for specimens of a particular age at the time of loading and a given condition of storage. Thus, Fig. 12

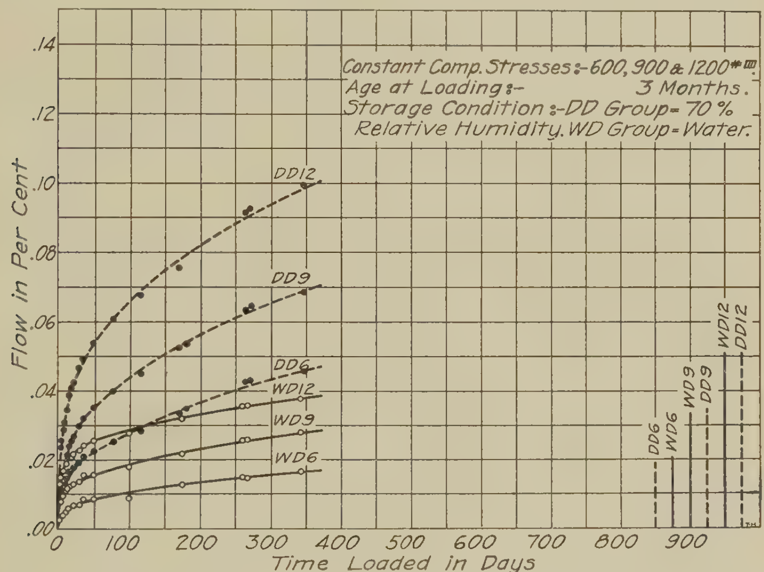


FIG. 11.—FLOWS OF GROUPS LOADED AT AGE OF THREE MONTHS (1926 SERIES).

is for specimens loaded at the age of two days and stored in water thereafter; and Fig. 17 is for specimens loaded at the age of three months and then stored in air. Each of the individual diagrams is a stress-strain curve for a particular instant of time. Thus, in Fig. 16, beginning with the one to the left, the curves show the stress-strain relation at the time the load was applied and after the respective intervals of seven days, one month, three months, and nine months of sustained load.

In order the better to show in more concise form the effect of the several variables in this series upon the stress-strain relation, the secant moduli for several stresses have been calculated from values taken from the curves of Figs. 12 to 17. These values are recorded in Table 7.

Consulting this table it is seen that under the action of a sustained compressive stress, other things remaining equal, the secant modulus is greater for the specimens stored in water; and this is true regardless of the age at which the load is applied. Thus for the specimens loaded at the age of three months, the secant modulus, after a nine months' period of sustained stress at 900 lb. per sq. in., is 1,500,000 lb. per sq. in. for water storage and only 920,000 lb. per sq. in. for air storage.

Also it is clear that the value of the secant modulus for a given stress, age at time of loading, and conditions of storage, varies materially with the length of period of sustained stress. Thus, under a stress of 600 lb. per sq. in. for those specimens which were loaded at the age of three months and stored in water, the secant modulus dropped in a nine-months' period from 3,200,000 to 1,700,000 lb. per sq. in., and the secant modulus of the corresponding specimens stored in air dropped from 3,200,000 to 980,000 lb. per sq. in.

TABLE 6.—FLOW AFTER ONE YEAR (1926 SERIES).

Group	Age at Time of Loading	Unit Stress, lb. per sq. in.	Days Loaded	Unit Flow, inches	Unit Elastic Deformation, inches	Total Unit Deformation, inches	Ratio of 5 to 6
1	2	3	4	5	6	7	8
WA2.........	2 days	200	357	0.00018	0.00032	0.00050	0.56
WB3.........	7 "	300	364	0.00017	0.00018	0.00035	0.95
WB6.........	7 "	600	364	0.00057	0.00082	0.00139	0.65
WC3.........	28 "	300	400	0.00011	0.00010	0.00021	1.10
WC6.........	28 "	600	400	0.00031	0.00044	0.00075	0.71
WC9.........	28 "	900	400	0.00052	0.00074	0.00126	0.70
DC3.........	28 "	300	396	0.00027	0.00009	0.00036	3.00
DC6.........	28 "	600	396	0.00055	0.00028	0.00083	1.97
DC9.........	28 "	900	396	0.00090	0.00062	0.00152	1.45
WD6.........	3 mo.	600	390	0.00016	0.00021	0.00037	0.76
WD9.........	3 "	900	390	0.00028	0.00033	0.00061	0.85
WD12.........	3 "	1200	390	0.00037	0.00051	0.00088	0.72
DD6.........	3 "	600	395	0.00046	0.00019	0.00065	2.42
DD9.........	3 "	900	395	0.00068	0.00035	0.00103	1.95
DD12.........	3 "	1200	395	0.00099	0.00050	0.00149	1.98

It is instructive to observe the effect of age at time of loading upon the secant modulus after a period of sustained load. The table shows that at a stress of 300 lb. per sq. in., continued over a nine-months' period, specimens stored in water having ages at time of loading of 7 days, 28 days and 3 months possess the respective secant moduli of 870,000, 1,500,-000 and 2,000,000 lb. per sq. in. The corresponding values for a stress of 600 lb. per sq. in. are 510,000, 1,100,000 and 1,700,000 lb. per sq. in.

8. *Summary and Conclusions.*—This paper gives the results of tests to determine the flow of concrete under sustained compressive stress. These tests have been carried on in the Materials Testing Laboratory of the University of California during the past two years and are still in progress.

The experimental work has consisted of measuring the changes in length of plain concrete cylinders subjected to a constant compressive stress, the load being sustained by means of car springs. (Fig. 2.)

The tests are divided into two parts: (1) the 1925 series, for which the specimens were divided into groups varying as to richness of mix (1:4

TABLE 7.—SECANT MODULI AT VARIOUS STRESSES (1926 SERIES).

Age at Loading	Period of Sustained Load	Stress, lb. per sq. in.	Air Storage Secant Modulus, lb. per sq. in.	Water Storage Secant Modulus, lb. per sq. in.
2 days	0	200	....	590,000
	7 days	200	....	440,000
	9 mo	200	....	400,000
7 days	0	300	....	1,500,000
	7 days	300	....	1,100,000
	1 mo	300	....	970,000
	9 mo	300	....	870,000
	0	600	....	940,000
	7 days	600	....	590,000
	1 mo	600	....	550,000
	9 mo	600	....	510,000
28 days	0	300	3,800,000	3,800,000
	7 days	300	2,000,000	2,700,000
	1 mo	300	1,400,000	2,300,000
	3 mo	300	1,000,000	1,900,000
	9 mo	300	850,000	1,500,000
	0	600	2,200,000	2,200,000
	7 days	600	1,400,000	1,500,000
	1 mo	600	1,100,000	1,300,000
	3 mo	600	860,000	1,200,000
	9 mo	600	760,000	1,100,000
	0	900	1,500,000	1,500,000
	7 days	900	990,000	1,000,000
	1 mo	900	800,000	950,000
	3 mo	900	680,000	870,000
	9 mo	900	610,000	820,000
3 mo	0	300	4,300,000	4,300,000
	7 days	300	2,100,000	2,700,000
	1 mo	300	1,800,000	2,300,000
	3 mo	300	1,400,000	2,100,000
	9 mo	300	1,100,000	2,000,000
	0	600	3,200,000	3,200,000
	7 days	600	1,900,000	2,500,000
	1 mo	600	1,600,000	2,100,000
	3 mo	600	1,300,000	1,900,000
	9 mo	600	980,000	1,700,000
	0	900	2,600,000	2,600,000
	7 days	900	1,700,000	2,100,000
	1 mo	900	1,400,000	1,900,000
	3 mo	900	1,200,000	1,700,000
	9 mo	900	920,000	1,500,000

to 1:7) and gradation of aggregate (fineness modulus 3.52 to 6.60) but all loaded at the age of about seven months to a stress of 640 lb. per sq. in.; and (2) the 1926 series, for which all specimens were of the same character of concrete but conditions were varied as regards age at time of loading (2 days to 3 months) and intensity of sustained compressive stress (200 to 1,200 lb. per sq. in.). (See Tables 1 to 4 and Art. 3.)

For both series, part of the specimens were stored in air at 70 deg. F. and 70 per cent relative humidity. For the 1925 series, the remainder of the specimens were stored in water spray at 70 deg. F.; for the 1926 series, the remainder were stored under water at 70 deg. F. (Figs. 4 and 5.)

The specimens of the 1925 series were under observation for five months after loading. Thereafter they were released from stress; and to each one load was gradually applied, the stress-strain relation and ultimate compressive strength being determined. (Table 5.)

The specimens of the 1926 series have been under observation for a year since loading, and during this time the loads have not been released. (Table 6.)

Observations upon control specimens, to which no load has been applied, have, in general, furnished data by means of which volumetric changes in the loaded specimens due to causes other than stress could be separated from the flow proper. An exception is the 1925 series stored in water spray, for which there were no control specimens.

For each specimen, the compressive deformation taking place immediately upon application of the load, which is here called the *elastic deformation*, has been observed. Periodically thereafter changes in length from that observed immediately upon the application of the load are determined. The change from the length when load was first applied, corrected for deformations due to other causes than stress (determined by observing control specimens), is here termed the *flow*. It is the plastic deformation in the direction of stress which is produced by the sustained load.

The following conclusions regarding the behavior of concrete under sustained compressive stress are based upon the observations made to date. Since the tests are still in progress and there is evidence that flow is in many cases still continuing, the quantitative values given below are, of course, not likely to represent the ultimate values.

1. Other things being equal, the leaner the mix the greater the flow. Thus, after five months of sustained load in air, for concrete of fineness modulus 6.60 (Fig. 6a) the flow of the 1: 7 mix, amounting to 0.02 per cent, is double that of the 1: 4 mix; and, for concretes of fineness modulus 3.52 (Fig. 6b) the flow of the 1: 7 mix, amounting to 0.05 per cent, is three times that of the 1: 4 mix.

2. Other things remaining equal, the smaller the fineness modulus, the greater the flow. Thus for a 1: 7 mix after five months of sustained load the flow of the group of fineness modulus 3.52 (Fig. 6b), amounting to 0.05 per cent, is double that of the group of fineness modulus 6.60 (Fig. 6a).

3. It appears that, for the same consistency of mix, variations in gradation of the aggregate, as measured by the fineness modulus, have a relatively greater effect upon the magnitude of the flow when the mix is lean than when it is rich. Thus for the 1: 7 mixes the ratio of the flow of the DL2 group (Fig. 6b) to that of the DL1 groups (Fig. 6a), at the end of five months' loading, is 2.0; while the corresponding ratio for the 1: 4 mixes is 1.3.

4. The flow of a given concrete stored in air is materially greater than that of the same quality of concrete when stored in a water spray, even though the conditions of curing are the same during the early life of the two concretes. Thus, for the 1925 series, all specimens were cured for the first two months under water, and thereafter those which were to be loaded in air were removed from the water. At the age of seven months the specimens were loaded. (See Table 2). The flow of the group of high fineness modulus and lean mix stored in air (DL1, Fig. 6a) is three times as great as for the corresponding concrete stored in water spray (WL1, Fig. 7a).

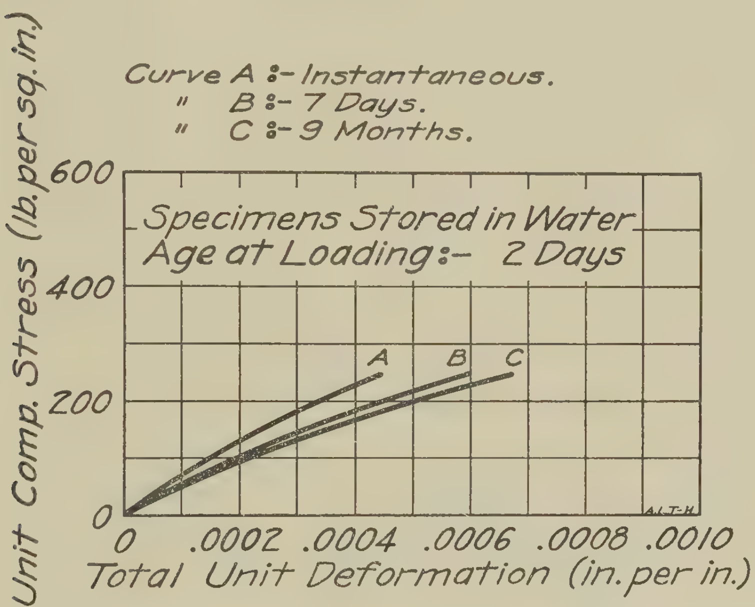
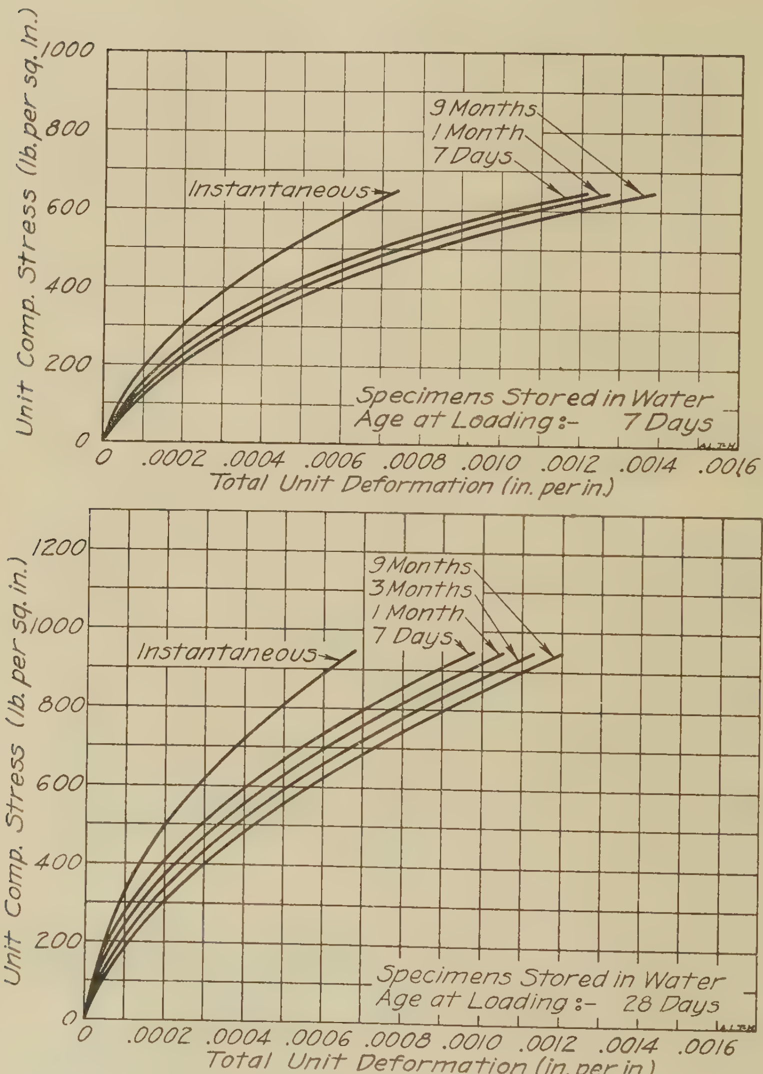


FIG. 12.

5. As between wet and dry concretes of the same character, in general, the flow appears to vary inversely as some function of the modulus of elasticity determined by dividing stress by elastic deformation, but the concrete of the higher ultimate compressive strength does not necessarily exhibit the lesser flow, nor does the concrete of the higher compressive strength necessarily possess the higher modulus of elasticity. Thus the concrete of 6.60 fineness modulus when stored in air at the age of a year has a strength of 4,540 lb. per sq. in. and a secant modulus (at 1,000 lb. per sq. in.) of 4,090,000 lb. per sq. in. The corresponding values for the concrete stored in water spray are 3,850 and 5,380,000 lb. per sq. in. (Table 5). The maximum flow for the concrete in air is 0.022 per cent and for the concrete in water spray is 0.007 (Figs. 6a and 7a).



FIGS. 13 AND 14.

6. Under the conditions of curing, loading, and storage to which the specimens of the 1925 series were subjected, the strength after the period of loading is lower for a given concrete stored in water spray than in air, but the modulus of elasticity (as determined by dividing stress by elastic deformation produced by a reapplication of load, is, in general, higher for the concrete stored in water spray (Table 5).

7. Other conditions remaining the same, the higher the sustained stress, the more rapid the rate of flow, and the greater the flow after a given period of loading. Thus, in the 1926 series those specimens loaded at the age of seven days and stored under water at the end of a year

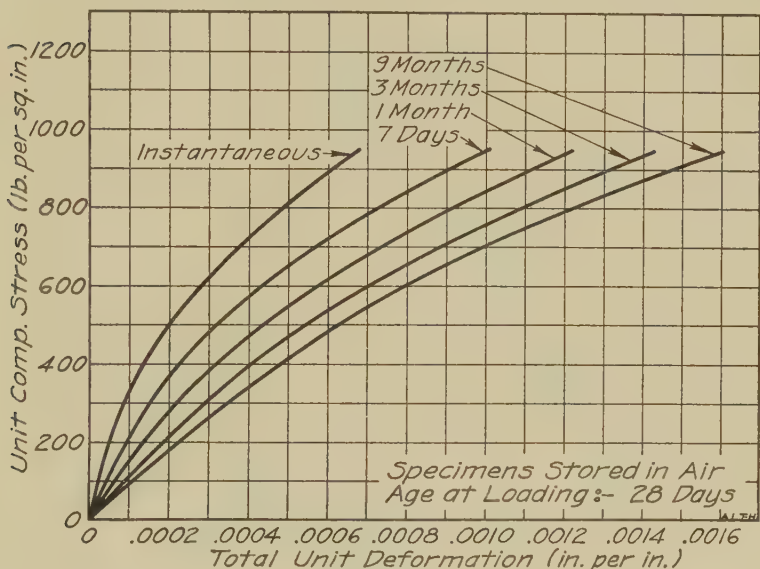


FIG. 15.

of sustained load of 600 lb. per sq. in. exhibited a flow of nearly 0.06 per cent, which is three times the flow produced by a stress of 300 lb. per sq. in. for the same period (Fig. 9). Again, those specimens loaded at the age of three months and stored in air under stresses of 600, 900, and 1,200 lb. per sq. in., sustained for one year, flow the respective amounts of 0.043, 0.070, and 0.100 per cent (Fig. 11).

8. Where specimens of concrete of a given character are cured in damp sand under identical conditions until loaded, those stored in air for a long period exhibit a materially greater flow than do those stored under water. The older the concrete at the time of loading, the greater the *relative* difference between flow accompanying air storage and that accom-

panying water storage. Thus those specimens loaded at the age of 28 days, after one year in air under a stress of 900 lb. per sq. in., flowed 0.09 per cent while the corresponding specimens under water flowed but 0.05 per cent (Fig. 10). The ratio of the former to the latter is 9/5; the corresponding ratio for specimens loaded at the age of three months is 7/3 (Fig. 11).

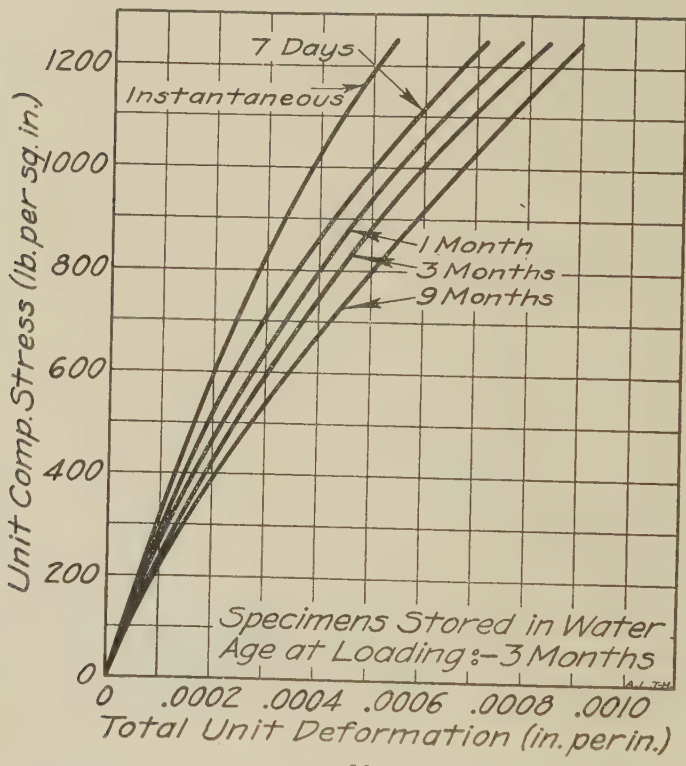


FIG. 16.

9. The older the concrete at the time of loading, conditions as regards manner of curing and magnitude and length of period of sustained load remaining constant, the less the flow. Thus, under a stress of 600 lb. per sq. in. sustained for a year under water, the flows taking place in the specimens loaded at 7 days, 28 days, and 3 months, are respectively 0.057, 0.030, and 0.017 per cent (Figs. 9, 10, and 11).

10. Under the action of loads sustained for one year, within the limits of the experiments as regards age at loading and range of compressive stresses, the ratio of the flow to the elastic deformation at the time of

applying the load, is greater than one for specimens in air storage but, in general, is less than one for specimens in water storage. Also, the indications are that the lower the stress, the larger the ratio of flow to elastic deformation (Table 6).

11. Considering the magnitude of the flow as influenced by the length of the period from the time of application of load, it is found that, in general, the flow during the first seven days is greater than during the succeeding three months (Figs. 12 to 17).

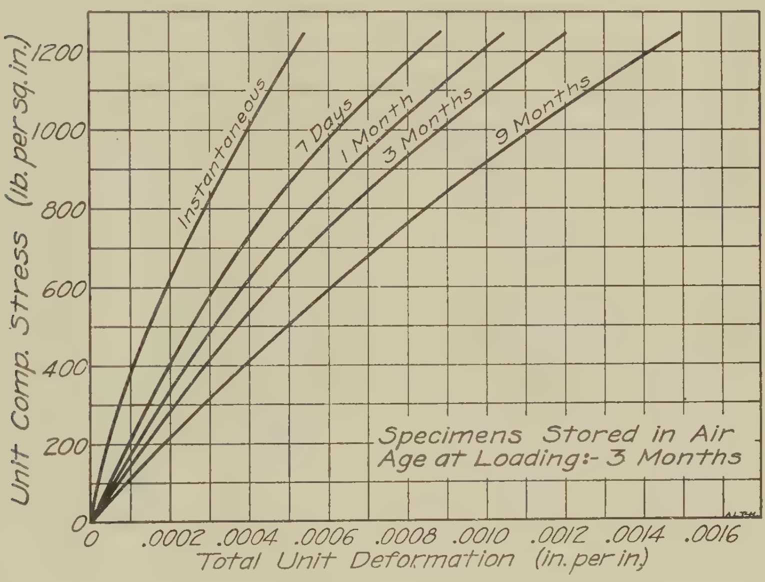


FIG. 17.

12. Considering the secant moduli as being determined by dividing unit stresses by corresponding total deformations, including both elastic and plastic, it is found that the secant modulus increases materially with time. Thus, for those specimens stressed to 600 lb. per sq. in. at the age of 28 days and stored in water, the secant modulus decreased from 2,200,000 lb. per sq. in. at the time of loading to 1,100,000 lb. per sq. in. after nine months of sustained load (Table 7).

13. Other things remaining equal, the secant modulus changes more rapidly when concrete is under sustained stress in air than when it is under water. Thus, for those specimens stressed to 600 lb. per sq. in. at the age of three months the secant modulus decreases from 3,200,000 lb. per sq. in. when the load was first applied to 980,000 lb. per sq. in. after nine months of storage in air; but for the corresponding specimens under

water the secant modulus only decreased from 3,200,000 lb. per sq. in. to 1,700,000 lb. per sq. in. (Table 7).

14. The ratio of the secant modulus at a given stress at time of loading, to the secant modulus after a period of sustained load, appears not to

Age at Loading	Stress, lb. per sq. in.	Ratio of Elastic to Total Secant Modulus			
		Specimens in Air		Specimens Under Water	
		7 Days	3 Months	7 Days	3 Months
7 days	300	...	...	0.7	0.6
	600	...	...	0.7	0.6
28	300	0.5	0 2	0.7	0 4
	600	0.6	0 3	0.7	0.5
	900	0 6	0 4	0.7	0.6
3 months	300	0 5	0 3	0 6	0 5
	600	0 6	0 3	0.8	0.5
	900	0.7	0 4	0.8	0.6
Average	....	0.6	0 3	0.7	0.5

vary greatly, regardless of the magnitude of the sustained stress and regardless of the age at the time of loading. This is shown by the tabulation above, where for several ages at loading and stresses, are given values of the ratio just mentioned. By this table it is seen that on the average for seven days of sustained load, this ratio is 0.6 for specimens in air and 0.7 for specimens in water; while for three months of sustained load the corresponding ratios are 0.3 and 0.5.

Discussion.—Flow of Concrete Under Stress.

J. R. Shank.—Professor Davis and his associates at the University of California have performed a work of much value, and the paper here presented on the flow of concrete is a considerable departure into the field of investigations of quantitative values to be assigned to concrete, so that a designer may know what stresses he may have in his designs under varying conditions of time, materials and external agencies. Values for this property of concrete have been indicated in the past by such investigators as E. B. Smith, F. R. McMillan, W. K. Hatt, A. T. Goldbeck, S. C. Hollister and others. In some cases the indications were rather startling. In many cases it was shown that this property is of such a nature that it cannot safely be ignored. It is expected that a thorough knowledge of this may effect a considerable saving of material in many cases such as deflection stresses in columns of small moment of inertia carrying long span beams of large moment of inertia, or stresses in elastic arch rings. **Mr. Shank.**

The writer has been interested in this work for some time and has had occasion to look into some phases of it. The first attempt followed a suggestion by C. T. Morris, Professor of Structures at Ohio State University, that an attempt be made to find out how much the flow might compensate for the stresses set up in arch rings as a result of temperature changes. It appeared that the first thing to consider was some form of apparatus whereby it would be certain that the load would be held constant. Fig. 1 shows a development of an apparatus following the suggestion by Professor Morris. The specimen occupying the compression side of a beam carrying a gravity load met this requirement very nicely. The load on the specimen may be varied for different tests by adding to the weight. either by loading the pocket or by adding extra weight, or by shifting the supports. The amount of load may be determined by inserting a calibrated steel bar in place of the test specimens. This device is perfectly satisfactory excepting that each unit takes up floor space of such magnitude that it amounts to quite a severe limitation on the quantity of work that can be carried on at any one time.

It was apparent after making the first attempt that temperature and humidity would have considerable effect on the specimen. Accordingly on all subsequent work a record was kept of temperature and humidity and a specimen of the same batch as that of the test specimen was stored close by, and measurements were made on it simultaneously with those on the test specimen. Fig. 2 shows a record of temperature, humidity and measurements on the control specimen (Dry Idler) for the tests shown on Fig. 3. By means of this control specimen and a room at ordinary tem-

peratures and humidities, curves such as are shown on Fig. 3 are drawn through points that, when plotted, varied from the curves by little more than the expected error in reading the Berry strain gage.

The first attempt was on an ordinary concrete. A pair of specimens were made and one of them was loaded at 28 days, the other remaining idle. At the end of another 28 days the specimens were interchanged in order to get at the negative flow. Further interchanges took place every 28 days. There was no control specimen used so that the curves showed the effect of temperature and humidity changes, but they were comparable to each other. This series showed rather clearly that the age at loading was a factor of much importance. The curve for the 56-day specimen took on a shape similar, excepting in the first few days, to the 28-day specimen, the same ages of both being compared. The 56-day specimen curve showed less deformation than the 28-day specimen throughout the entire test and

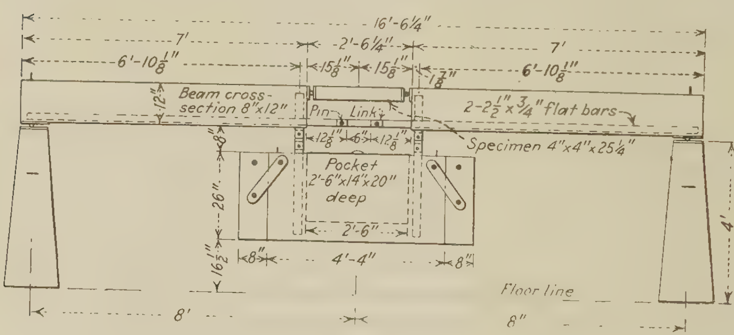


FIG. 1.—ASSEMBLY OF APPARATUS.

this difference was fairly constant. The later interchanges showed a stiffening of both tests, that is, there was less of both positive and negative flows at loading and unloading respectively.

The results of this preliminary test were interesting enough to lead to the ambitious attempt to thoroughly investigate this field in all of its phases even though it should cover a long period of time. No attempt was made to lay out the whole field, as it was not felt that enough was known. It was thought rather to start work and let the field open out, hoping to learn more about it as the work went on. At this time it is not possible to make up anything but a progress report, and indications rather than conclusions will be observed. The work of Professor Davis has given the occasion for adding observations made at Ohio State University to confirm his excellent work. Two phases have been studied so far: (1) the effect of age at loading; and (2) the effect of materials proportions. At present investigations are under way on some of our common building stones of a material similar to the stone used in the concrete. Tests are started on a comparison of limestone aggregate and silica aggregate. The work is

being now carried on under the Engineering Experiment Station which furnishes what funds are necessary for most of the materials and some of the labor. Acknowledgments are due to C. L. Lockett and Orville Seeger, E. L. Merkel and B. J. Merickel, E. P. Coady and H. M. Hughes and Werner Jung and O. N. Essex, undergraduates who have assisted in this work; and to the Marble Cliff Quarries Co., Columbus Ohio; Columbia Silica Co., Akron, Ohio; Ohio Cut Stone Co., Cleveland, and the Indiana Limestone Co., Bedford, Indiana, for furnishing materials. Credit is also due Prof. C. T. Morris, who gave the original and many subsequent helpful suggestions, and to L. W. Jones, who worked up the graphs.

Fig. 3 shows the results of this first series. The upper set of curves plotted from a common zero load origin with the elastic deformation deducted, and the lower set of curves from a common age but with the elastic

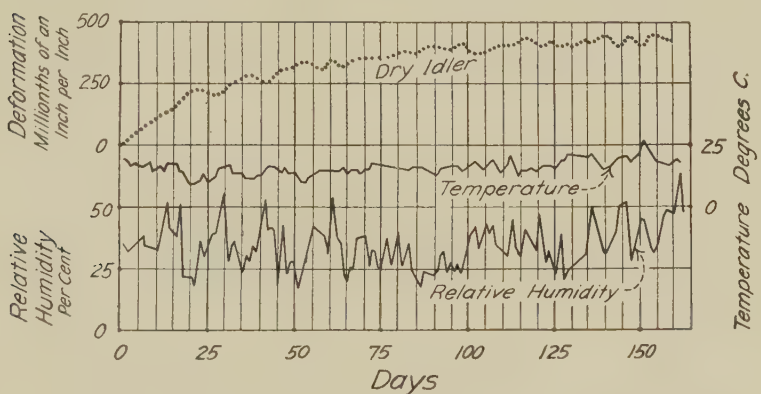


FIG. 2.—TEMPERATURES, HUMIDITIES AND UNLOADED SHRINKAGE.

deformation included. The lower set of curves is quite graphical in its showing of the similarity of all curves after the first one or two weeks of loading. An attempt was started to obtain formulae for these curves and so far only one has been worked out: that shown for the seven-day test on the lower set of curves where x is millionths of an in. per in. and t is days. This formula fits the curves very well at all points above the initial load deformation. The question came up as to the effect of the different ages of loading on the modulus of elasticity and the upper set of curves shows the loads removed at the same age for all specimens. The values for E varied from 3,050,000 to 3,360,000 English units with an average of 3,260,000. Fig. 4 shows the initial moduli as well as some of the data concerning the specimens.

Oscar Faber, M. Inst. C. E. in Paper No. 4645, "Plastic Yield, Shrinkage, and Other Problems of Concrete, and Their Effect on Design," now under consideration by the Institution of Civil Engineers, has made some investigations on flow and some calculations on the use of data now avail-

able as to its effect on the stresses in concrete structures. He has coined the term "factor of plasticity" which he defines as "the ratio of eventual strain to original strain" which he has designated as k. This idea is very

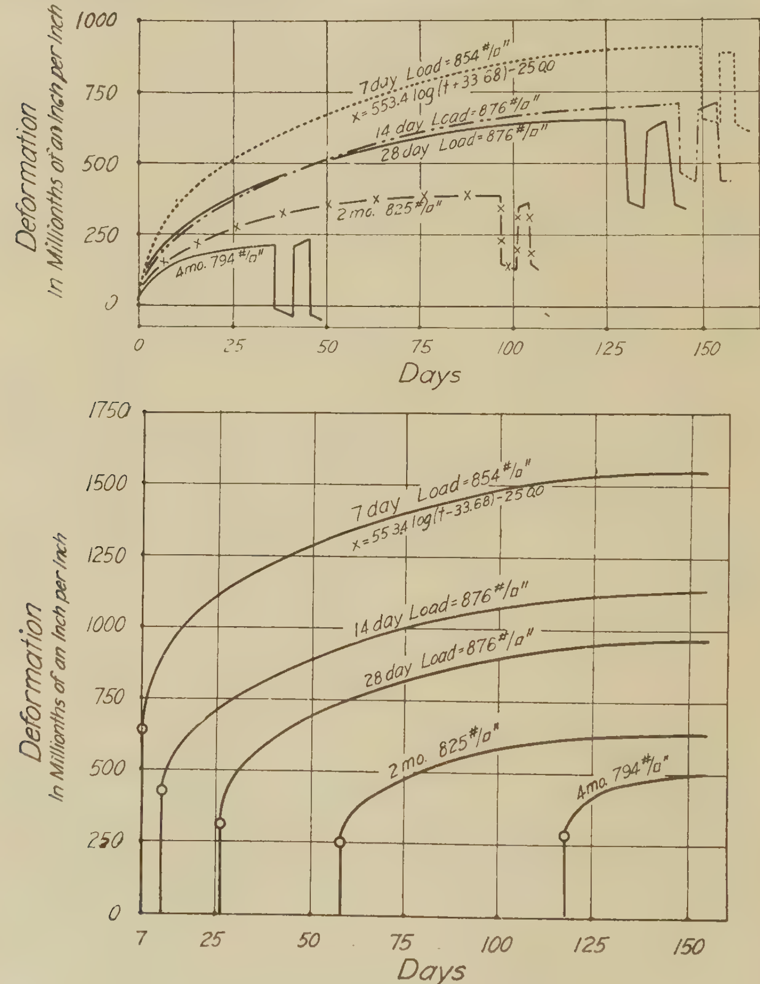


FIG. 3.—EFFECT OF AGE AT LOADING.

good, but it appears that American practice will have to assign some other letter to it, such as r, because k has its own well-established meaning here. He uses r as a coefficient to use with our term n the ratio of the moduli of elasticity. He would state $\frac{P}{[A + (nr - 1)\ As]}$ for a column having been

loaded for some time. With Mr. Faber's idea in mind, the work shown on the upper set of curves, Fig. 3, was replotted on Fig. 4, using days as abscissae and $r - 1$ or factor of plasticity minus one as ordinates with the idea of determining whether this might be constant for all ages of loading. It appears that it is not. Just why it is not and the laws that govern the variations are problems yet to be worked out by further tests.

The following year, 1926-1927, an attempt was made to look into the effect of proportions of materials on the flow. Figs. 5 and 6 show data concerning these tests in which it should be noted that two of them BII and CI are mortars. Fig. 5 shows the regular plot after corrections had been made for temperature and humidity by means of the control specimens. These specimens were loaded in proportion to their respective strengths

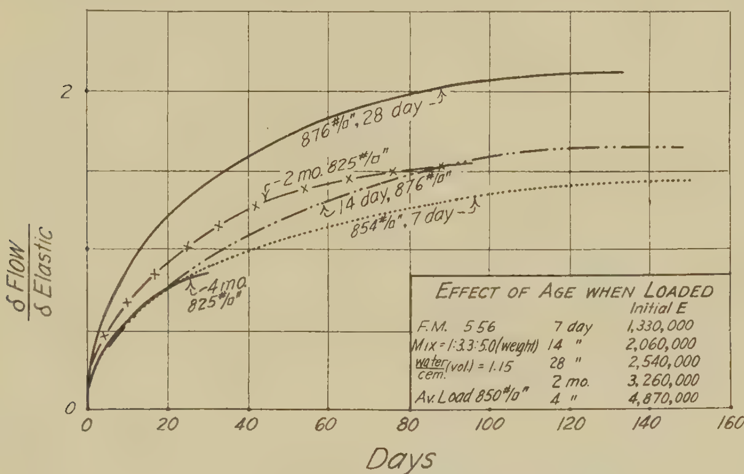


FIG. 4.—FACTOR OF PLASTICITY—EFFECT OF AGE AT LOADING.

as shown by the 28-day compressive tests strengths; they were also proportional to those of the specimens of the year before. This was done because it was expected that a designer using high strength concrete would give it high loads, as specifications are usually written for allowable unit stresses. Fig. 5 shows the effect of a number of factors, one of which is the different unit stresses. Fig. 6 illustrates these same curves with the effect of the difference in unit load eliminated on the assumption that the flow was proportional to stress. It may be noted that the curves for CI and FI, one of them a 1 : 3 mortar and the other a 1 : 3 : 6 concrete, coincide throughout their lengths. As a basis of comparison the others are expressed in percentage of these two at 60 days. The tabulation of fine aggregate divided by cement by weight expressed in per cent of that for CI and FI shows curves for DI at 60 days falling in its proper place with BII and EIV curves running 38 per cent and 30 per cent high, respectively. Though

this deduction is somewhat complex, it appears that it might be worth while to investigate further whether or not the flow is a function of the mortar mixture.

Fig. 5 shows a tryout of one of the A specimens, the one that served as the 28-day specimen as shown in Fig. 3. This second loading was made at an age of 461 days. It will be noticed that the flow still exists but is

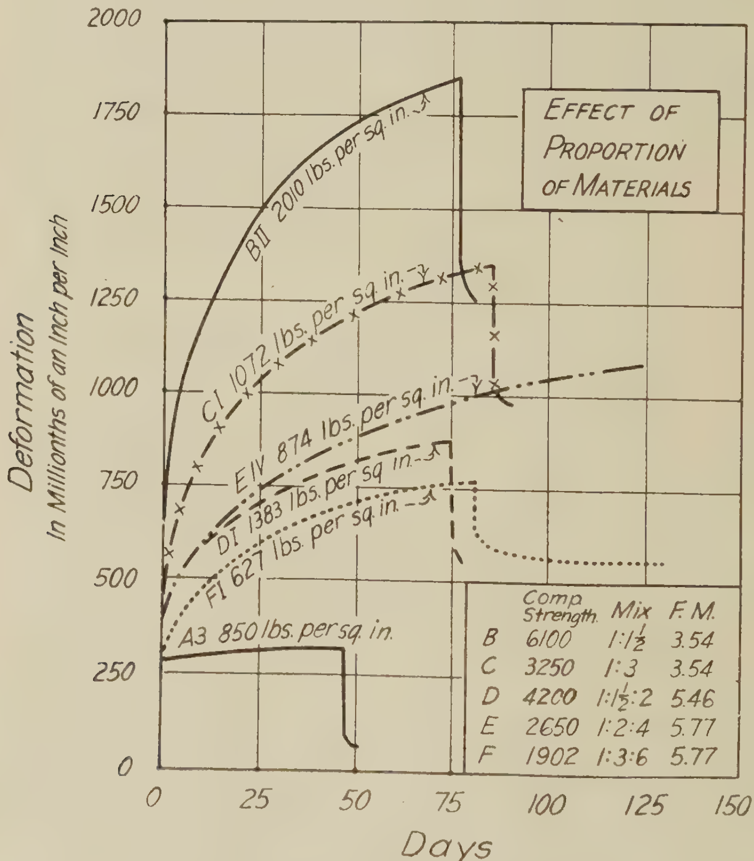


FIG. 5.—EFFECT OF PROPORTIONS OF MATERIALS.

very slow. There was no smooth curve coming off from the elastic deformation, but instead a sharp angle indicating that there was no short time flow. This indicates that flow cannot be given much consideration as compensating for the temperature stresses in arch rings.

Fig. 7 is the 1926-1927 series plotted as was Fig. 4 for the 1925-1926 series. Here for vastly different mixtures and loads all placed at the same

age, the agreement is much better. The variations here shown do not exactly check the last statement of conclusion No. 10 of the report, if concrete and mortars are considered together. It does, however, for concrete alone or mortars alone. As for the effect of age at loading, Fig. 4 of this discussion indicates an advance in the amount of flow to 28 days and then a recession to 2 months and a farther recession to 4 months.

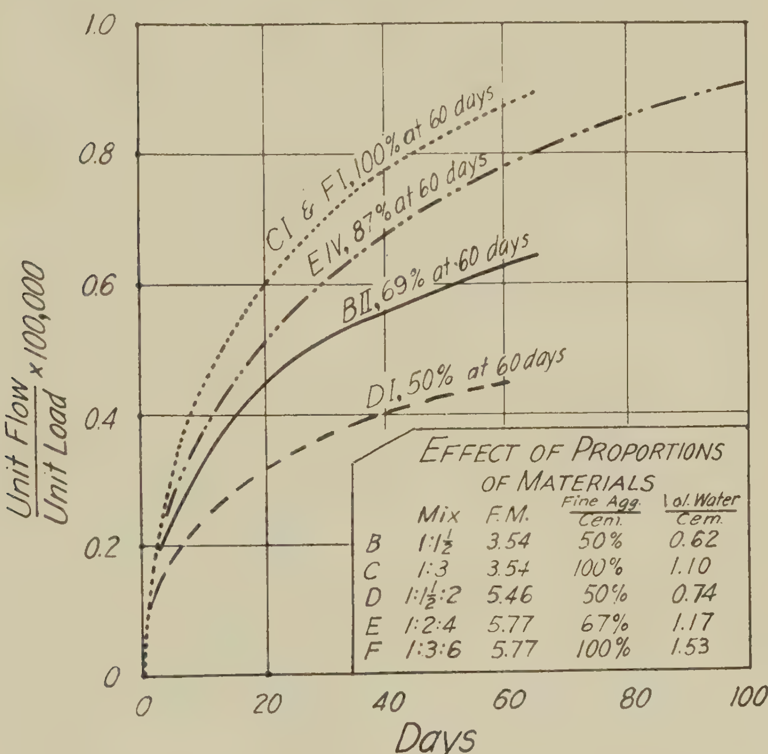


FIG. 6.—EFFECT OF PROPORTION OF MATERIAL—COMMON UNIT LOAD BASIS.

Referring to conclusions Nos. 1 and 2 of the report, the discussor is not at variance with the report, but it appeared that if each specimen or set of specimens had been loaded in proportion to its compressive strengths, on the assumption that the flow is proportional to the load, curves for DL1, DR2 and DR1 would have been nearly the same curves. Multiply the compressive strengths of each of these by the percentage flow at, say, 60 days and you get 43,400 for DL1, 42,860 for DR1 and 45,800 for DR2. DL2 would give much more than the others, 71,300. Not considering DL2, it would appear that for a given load the flow is inversely proportional to the

strength. DL2 is so much at variance with the other three, that, unless some reason, such as density, could be found for its high flow, the observation just stated would have to fall considerably in value. A low density would be equivalent to smaller cross section and therefore a higher unit stress. Fig. 5 of this discussion shows a fair agreement of the three concretes, but the mortars seem to be in a class by themselves. An investigation into the density factor, that is, the sum of the absolute volumes of

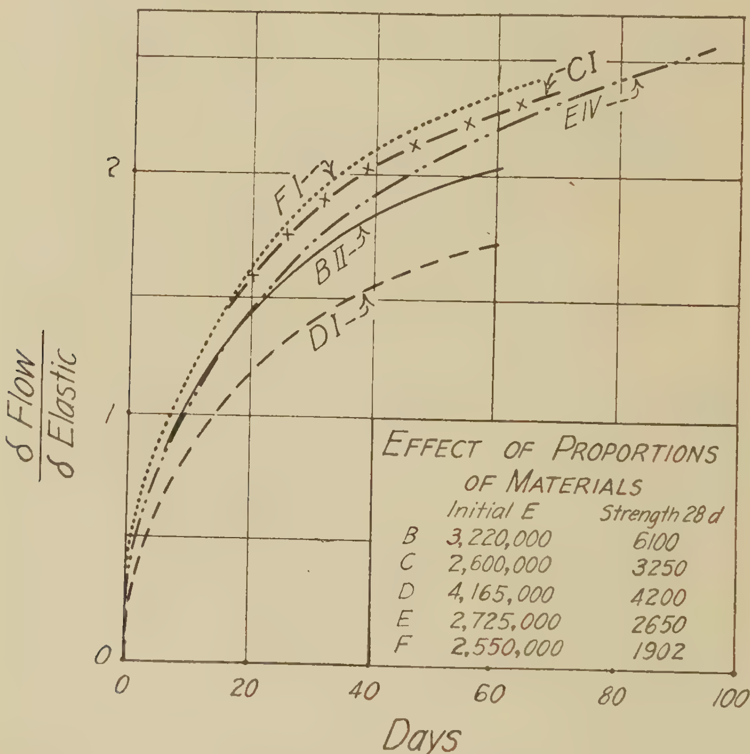


FIG. 7.—FACTOR OF PLASTICITY—EFFECT OF PROPORTIONS OF MATERIALS.

the solid materials over the external volume show specimen EIV with a density of 0.797, DI of 0.818 and FI of 0.824. DI is 78 per cent of the difference between EIV and FI. BII and CI have densities of 0.742 and 0.731, respectively.

Figs. 8, 9, 10 and 11 of the report show excellently the effect of age at loading. If a 600-lb. per sq. in. stress be considered, at 50 days for 7 days' loading we have 0.048 per cent flow, for 28-day loading, 0.037 per cent flow, and for 3 months' loading 0.023 per cent flow. The upper curves of Fig. 3 of this discussion show 0.0675 per cent flow for 7 days' loading,

0.051 per cent flow for 28 days, 0.037 per cent flow for 2 months and approximately 0.023 per cent flow for 4 months. If these latter were reduced by the ratio of the loads there would follow 0.0473 per cent flow, 0.035 per cent flow, 0.027 per cent flow and 0.0175 per cent flow, respectively. The average of the last two would be about 0.022 per cent flow which might be considered as an approximate result for a 3 months' loading. These results check remarkably well: 7 days 0.048 per cent vs. 0.047 per cent; 0.037 per cent vs. 0.035 per cent; and 0.023 per cent vs. 0.022 per cent.

The discussor is much interested in looking into the question of whether the amount of flow at any given period after loading may not depend upon the proportion of the load to the strength when loaded. It would be interesting to look into this if the strengths at times of loadings are available for the concrete of the data shown on Figs. 8, 9, 10 and 11. The similar comparison made on the data of the upper curves of Fig. 3 of this discussion showed that where the load over strength increased the flow increased and where the ratio decreased the flow decreased but the proportions were not nearly exact. It is expected that varying conditions of humidity would have considerable effect.

Fig. 5 and the upper curves of Fig. 3 of this discussion show that in all cases the flow in the three months following the first 7 days was considerably greater than that of the first 7 days which is not in accord with the conclusion in No. 11 of the report. This greater flow after the first 7 days may possibly be due to the fact that the atmosphere at Ohio State University was considerably dryer than that of a relative humidity of 70 per cent. It is entirely probable that the amount and duration of the flow is greatly affected by the humidity of the atmosphere.

It appears that the most definite and best established conclusion of this report is that of No. 4 that the moisture content in a concrete is of prime importance when considering the flow. Another of much value is that the older a concrete is when it is loaded the less the flow. The effect of properties of the concrete itself and the amount of the load seem to be of so complex a nature that only with many very carefully controlled specimens can it be hoped to establish any definite rules and quantitive expressions.

Concrete Roofing Tile Problems.

By Leslie H. Allen.*

The use of concrete roofing tile is steadily growing, particularly in Chicago, New York, Los Angeles, Pittsburgh and Detroit, where plants equipped with automatic power machines are turning out by quantity production methods a tile of high quality, maximum density, strength and imperviousness, fully equal and in some respects superior to the competing clay product. There is as much difference today between the modern steam-cured roofing tile made on automatic machines and the hand-made product as there is between the present-day concrete block of A. C. I. standard and the old hand-made back-yard variety.

Certain problems, however, still beset the roofing tile manufacturer and engage his earnest thought and study. The most pressing of these is color. In the early days much cheap red mortar color was used with disastrous results. The cheaper red oxides often contain upward of 50 per cent of impurities and some of these, being soluble, leach out on the surface of the tile; others oxidize on exposure and leave black stains. The early methods of applying color were not effective and much of the hand-made tile produced eight or ten years ago has now almost entirely lost its color and remains a dull cement gray, slightly stained with red.

To produce red colored tile, it is now the general practice to use either a precipitated ferric oxide C. P., containing at least 95 per cent Fe_2O_3, or a roasted haematite ore found in deposits of exceptional purity in Spain, which analyzes about 85 per cent Fe_2O_3. The latter color is a darker, duller shade than the chemically produced red, but it is satisfactory to those who prefer a dull red roof, as many do, and is very much cheaper.

The architect or builder expects a much deeper, richer color in roof tile than he does in trim stone or stucco. It must have at least as rich a color as its clay competitor, and to attain this result, proper methods of surface coloring must be used. It is not possible to mix sufficient color in the body of the tile to produce the required shade without seriously reducing the strength of the concrete, and moreover, the expense of doing so would be prohibitive.

The old method of coloring by hand consists of mixing dry cement and color together, dusting it over the face of the newly formed tile while wet, and rubbing it in with a steel trowel. It is usual to grind the cement and color together in a small ball-mill in order to insure thorough mixing.

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(There is no other advantage gained by this grinding, as the colors now on the market are already ground to two or three times the fineness of portland cement, but the regrinding seems to be the most effective way of obtaining a thorough mixture of cement and dry color.)

The result, however, is not satisfactory, as the cement is not properly hydrated and is not properly bonded with the base; the only water that it receives is absorbed from the base, and this, being a dry tamped product, has not sufficient water for thorough hydration and has none to spare for hydrating the additional cement in the top surface. As a result, the surface dries off before it is properly hardened and the excessive troweling needed to bond the colored surface to the base brings laitance to the surface. In the course of two or three years the surface begins to dust off and in 7 or 8 years is nearly all gone.

The newer method consists in thoroughly mixing cement and color and fine sand with an excess of water in a specially designed batch mixer. Mixing is continued for at least ten minutes and the liquid mixture is spread on the face of the newly formed tile as it passes through the automatic machine. The excess moisture is immediately absorbed by the tamped concrete in the body of the tile so that the top surface is bonded, both by suction and by cohesion to the base and has a correct cement water ratio. This method gives an impervious surface of extreme hardness and density which has stood the test of several years of exposure without any of the dusting or crazing found in the dry colored hand-made product.

The manufacturer of hand-made tile has been forced in self-defense to put a little color into the body of the tile so that when the top surface had dusted away there was still a red tint left in the body, although this was nothing like so rich in tone as the face. With the new method this is no longer necessary.

In the selection of color pigments it is not only important that they be pure, but also that they should not contain elements that react with the alkalis in the cement and change the color. Many colors on the market are not proof against alkali reactions; and some, such as those containing lead or cadmium combine with the alkalis in the concrete to form salts which will not combine with the cement but are deposited in the form of a fine powder on the surface of the tile, and can be brushed off with the finger.

For red and yellow colors, various grades of ferric oxides are satisfactory. For green, the only satisfactory color is chromium oxide Cr_2O_3. The cheaper greens, sold as chrome green, are mixtures of Prussian blue (potassium ferro cyanide) and chrome yellow (lead chromate). The reaction of this color with cement destroys the blue, leaving a bright yellow color only.

Brown colors are compounds of ferric oxide and manganese dioxide and other inert constituents. I have not found a brown that was entirely satisfactory, as manganese dioxide immediately oxidizes, giving the finished tile a much darker tint than the original pigment; however, on exposure to the weather this tint slowly changes color, probably due to the dis-

appearance of the manganese dioxide after oxidization, leaving a permanent brown color in the tile that is much lighter in tint than the original pigment. As brown is mostly used in color blends, this change in color is not usually noticed and the final color is a very pleasing one. But as it is not possible to foretell the exact final shade of the product, it is not wise to sell roofs of uniform brown color unless the purchaser is forewarned that the final color of the roof will be much lighter in tint than the tile shipped to the job.

Ultramarine blue gives good results in making blue tile. White tile are colored with white cement and white silica sand. It is not possible to produce a jet black tile, but a tile that approximates black in color can be produced with black ferrous oxide Fe_3O_4, and dark shades of other colors are made by mixing various percentages of black oxide with the red, green and other colors mentioned above. Several variations of green color can be made by mixing small amounts of blue, brown, yellow oxide with the green.

The use of lamp black and other carbon blacks is not advisable in coloring concrete roofing tile or any other concrete products. The lamp black does not combine with the cement; most of it seems to float to the surface and is there deposited and dusts off in a very short time. It is hard to keep lamp black from getting lumpy, and small pellets of black will spot and disfigure the surface of the concrete.

Efflorescence is a serious problem in the roofing tile plant, as a small amount of efflorescence is far more conspicuous against the intense colors of the tile than it is in trimstone or stucco of less vivid hues. Efflorescence is due to the presence of soluble alkalis in the concrete. These are drawn to the surface by evaporation after the tile has become wet and are deposited on the surface and eventually washed off by rain. The amount of alkali in the thin section of the tile is never very large and even under the worst conditions it disappears in two years at the most, and in most cases three months' exposure to the weather is sufficient to remove it. Efflorescence can be largely controlled by extreme care in curing. It is important that the newly formed tile should not be subjected to sudden changes in temperature and should not be allowed to dry out before they are thoroughly hardened.

It is not practical to steam cure tile while they are still on the pallets, but the tile should be kept at a temperature of about 60 deg. for the first 24 hr. at which time they are hard enough to be removed from the pallets and stacked on edge. They should then be placed in a steam curing room and cured in warm, moist air. The temperature should be slowly raised to 90 deg. and the tile should be kept in this room for at least 12 hours. When the steam curing period is over, the curing room should be allowed to slowly cool down to outside temperatures before the tile are moved. They should not be brought suddenly into the open air.

It is probable that this method of treatment prevents any evaporation from the surface until the surface has hardened, and small capillaries are not formed by evaporation from the top surface as they would be if the

tile were suddenly dried. This reduces the possibility of evaporation through the top surface and results in most of the efflorescence being found on the back of the tile where it does not show.

Efflorescence can be checked not only by care in curing, but also by the selection of clean materials and by care in making the top surface of the tile denser than the base. This cannot be done by hand methods. Experience has shown that some brands of cement cause more efflorescence than others and in some districts the sand and water also contain soluble material which will add to the efflorescence. In one plant which I visited, unwashed sea sand had been used with disastrous results.

As in other concrete products the strength of the tile is affected by the quality and grade of the sand. It is very difficult to obtain good sand suitable for roofing tile manufacture. An ideal sand should contain about 10 per cent of grits passing ⅛ in. mesh and be uniformly graded down from that to the finest sand, with a slight excess of fine.

The steel or cast iron pallets on which tile are made have to be oiled in order to prevent concrete from sticking to the pallets. Many manufacturers have used crude oil for this purpose, but as the concrete absorbs some of the oil, this practice is not a satisfactory one. The sulphur compounds in the oil are drawn to the surface and there oxidize, causing black stains which greatly disfigure the surface of the tile. The only lubricant that we have found satisfactory is a mixture of animal tallow and kerosene, which gives very good results.

Most roofing tile have a thickness of about ⅜ in. and as they are made without reinforcement it is very important that they should be of maximum hardness before shipment, as otherwise they are subject to considerable breakage. It is our practice to keep tile in storage for at least 30 days after steam curing in warm weather, and for 60 days or more in colder weather.

In shipping by truck or freight, the tile must be stacked on edge in the truck or car and plasterer's lath placed between each row to prevent jarring on the journey. They are stacked on the ground in the same manner.

The application of concrete tile is very similar to the application of clay tile, but the roofer has one additional problem to face in the cutting of hips and valleys. Several methods of cutting have been tried, the most satisfactory being the use of the standard slater's hammer and stake. An experienced mechanic can cut tile to straight lines more quickly and easily with this than with pincers or cutting machine.

The common method of placing concrete tile is to lay them on wood strips, the tile having lugs that hook over these strips. The tile interlock and are thus held securely without the use of nails. The additional cost of wood strips is more than offset by the saving in labor and the saving in cost of copper nails, so that the total cost of laying is somewhat less than the cost of laying a nailed tile. Some concrete tile manufacturers, however, are putting nail holes in their tile, being apparently forced to do so by the adverse sales talk of vendors of competing materials. In so doing

they are discarding the big advantage of the interlocking method of laying. I have never found that if the interlocking method of laying were properly presented by a salesman that it proved any hindrance to the sale of the tile.

The steady growth of the business, coupled with the increasing demand for permanent roofs, encourage us to believe that eventually this product will dominate the roofing field.

Discussion.—Concrete Roofing Tile Problems.

Maximilian Toch.—This paper is exceptionally opportune. It would appear that Mr. Allen knows what he is talking about and any assistance any member of this Institute can render should be frankly done. The problems that Mr. Allen mentions which still beset the roofing tile manufacturer appear to be that of color more than anything else. **Mr. Toch**

At the Chicago meeting, February 24-27, 1925, I presented a paper called "Shall Anything Be Added to Portland Cement," in which a fairly complete analysis was given of the tensile strength produced by various pigments, and in which was shown that the red shades, which contain calcium sulphate or gypsum, reduce the tensile strength to such an extent that a tile cannot possibly last when made of a material of that type.

Furthermore, many of the adulterants in ordinary pigments are water soluble, and calcium sulphate is no exception. Efflorescence, excrescence and in some instances laitance is caused by the exudation of a soluble portion of a color, and I heartily agree with Mr. Allen when he says that an 85 per cent ferric oxide or 95 per cent precipitated ferric oxide are the proper materials to use, provided the balance in any case is silica or insoluble silicates.

In order to obtain deeper and richer color effects, like Burgundy reds and maroons, which are very much in vogue, some of the organic lakes can be added and they will produce very beautiful effects, but the price of the pigments mitigates against their wholesale use.

It is notorious that the ultramarine and cobalt blues when they are free from alum and free from calcium sulphate, increase the tensile strength of concrete tremendously. Looking over the work of our laboratory, I note that when nothing was added to the cement, an average tensile strength is produced of 435; the pure blues produce a tensile strength of 635. When these blues are mixed with some of the red oxides, very beautiful purple and maroon shades can be made.

Chrome Greens, which are made of prussian blue and chrome yellow, should not be used for two reasons. The first is that any alkalinity will destroy the prussian blue and turn it brown, and any lead chromate mixed with cement weakens the cement very materially. Therefore, there are only two greens that are available. One is the chromium oxide, opaque, and the other chromium hydroxide, which is transparent. The latter is, however, too expensive for ordinary cement purposes.

Other permanent greens can be made by mixing cobalt blue and cadmium yellow or cobalt blue and yellow oxide of iron. These are muddy, but sometimes these muddy shades are desirable.

The only reliable yellows to use for cement purposes are the hydrated oxides of iron and the cadmium yellows. Cadmium yellows as a rule are very strong, produce brilliant lasting shades, but they are expensive.

Browns are permanent and not difficult to make, as there are some beautiful browns which are made artificially from hydrated oxide of iron that approach maroon in color. Other browns can be made by mixing carbonaceous blacks with red or by mixing carbonaceous blacks with the hydrated oxide brown.

It is possible to produce a jet black tile by using a pigment which is composed of the black ferrous oxide precipitated on carbon black.

The colors which weaken portland cement and which should not be used are zinc chromate, carbonate of copper green, calcium sulphate reds, known commercially as Venetian Red, chrome yellows, light medium and orange, and some of the organic lakes. The latter, however, as a general rule are expensive and would naturally be omitted.

For brilliant colors, such as jade green, fire red, ox-blood red, mandarin orange and fancy colors of that type, it is quite possible to utilize these with excellent effect in the manufacture of tile, provided the purchaser is willing to pay the price.

As regards the top dusting and the efflorescence which is so likely to occur in tiles, this is very largely overcome by coating the tile after it is ready for market with an acid resin varnish of the China wood oil type. A thin solution of this material changes the refractive index of the tile so that it makes the color much more brilliant. Furthermore, it prevents efflorescence to a very marked degree. At the same time a tile so coated becomes rain-proof and does not absorb very much water.

Viewpoint of Architect and Engineer Regarding Concrete Products.

By George J. Eyrick, Jr.*

Your program committee has requested that the writer present to you a paper on a large subject in a very short time. In the first place, let us roughly classify concrete products into two general classes, (1) concrete stone as used for facing buildings and in general copying of natural stones, and (2) concrete tile, blocks, bricks, and similar units, commonly known as concrete masonry units. Such items as concrete pipe, piles, sheeting and the like, we will not discuss or consider, as they are primarily engineering units. The methods of manufacture will not be discussed at any time in this paper, as the men making these commodities know and have their own ideas on this subject, and I feel that this should be left for men in the business to talk over.

The reason for making this division is, that the average architect assumes that all concrete products are made in the same way, and that after seeing a wall of concrete block, he imagines that all concrete products will look the same. This is not as far fetched as it may seem, for not three months ago a well-known architect on residential work in Detroit asked me if I thought "artificial stone" was satisfactory for exterior work, and when I showed him samples of stone made by a nationally known manufacturer, he was very much surprised and said: "I didn't know it could be made as well as that."

Throughout this paper I will use the word architect as meaning both architect and engineer, because for the purpose of this paper both are interested in concrete products from practically the same viewpoint.

In spite of the fact that the architect does not favor substitutes, it is well for him and all others in the building industries to remember that this is an age of analysis and synthesis. All natural substances are being broken up into constituent elements, followed by the reproduction of the original substance in the laboratories by a synthetic process. In these processes we may, by changing the proportions or by adding new elements, obtain in the synthetic product new qualities or augment such qualities as we may wish to accentuate for specific reasons.

The production of concrete stone, frequently called artificial stone, cast stone, cast cement or cement stone, is closely allied with the synthetic process. Throughout this paper we will use the term concrete stone,

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for we use natural stone for the production of cement, and to emphasize this point I will quote Prof. George F. Swain's "Fundamental Properties of Materials," page 157:

> "It (portland cement) is defined as the product obtained by finely pulverizing the clinker produced by burning to incipient fusion an intimate and properly proportioned artificial mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum. Clinker is the material as it comes in lumps from the furnace. The main difference between natural and portland cement is that the former is obtained by burning a natural rock and the latter by burning an artificial mixture. The calcareous materials used may be pure limestone, chalk, or marl, or hydrated lime. The argillaceous materials may be clay, slate or shale, impure limestones containing clay, or blast-furnace slag. Knowing the chemical composition of the raw materials, they may be mixed in proper proportion to produce the silicates or aluminates required in the cement, leaving little or no free lime or free clay."

Anyone can easily recognize the products which are thus broken down for the manufacture of the cement. For an addition to this cement, to make our concrete stone, we add crushed stone of any sort,—gravel, crushed shells, sand, slag, in fact any of an almost limitless variety of aggregates or materials,—then by adding water we have a plastic mass which can be cast or worked into any shape or form. When the mixture hardens, the result is a hard stone, the color and form of which might have been varied at will and which may be as hard or harder than the natural stone whose place it is to take. The surface of this product may be treated or tooled the same as the natural stone. The biggest trouble is this,—the architect does not always realize that it is a synthetic product and not an imitation. It is the duty of the concrete stone makers to convince and show the architect that this is the case.

There are in every community, regardless of size, many examples of concrete stone which are a disgrace to the community as a whole. The writer can name many in Detroit, and examples of this kind are always referred to by the architect when he is approached by a representative of a concrete stone maker. This is one of his greatest objections. Much excellent work has been done with concrete stone, for example, buildings at U. S. Military Academy, West Point, N. Y.; Cram, Goodhue & Ferguson, architects; buildings at Boston College, Boston, Mass.; Maginnis & Walsh, architects; and the Players Building, Detroit, Smith, Hinchman & Grylls, architects.

For other than trim stone, the office building recently erected for the Portland Cement Association in Chicago, where the complete facings were of concrete stone, is a very fine example. Forget the word "imitation" in connection with your product and show what your product is and what it can do, and sell it as a synthetic product of definitely known qualities, capable of definite results, both physically and artistically.

There is still another angle to this subject and one which troubles the architect not a little. It is, "how shall I specify concrete stone and be reasonably certain that I will get the effect I demand in my project?" The architect has many subjects with which he must be familiar and cannot take the time to study methods of manufacture, technique of design

and finish of a new or comparatively new product. Why do not the concrete stone manufacturers get together and agree upon a standard specification and a set of details as has been done by the National Terra Cotta Society, the National Association of Marble Dealers, the National Building Granite Quarries Association, Inc., the National Terrazzo and Mosaic Contractors' Association, and many other similar organizations? The writer realizes that various manufacturers have different methods and processes for doing their work, but for the sake of the industry, why not get together so that the architect knows where he is and what he can expect? If he desires a limestone effect, let him be sure that he will get it, and that after he gets it in his building, it will stay limestone and not be a piece of cement finish, full of cracks, crazing, checks, discolorations, spauled surfaces and what not. Just so long as you are unable to give the architect, the engineer and the contractor such assurance and service, just so long you are going to have difficulty in getting your product into the better offices, and so into general use. It is well to remember that the little fellow copies the big fellow and if the big fellow uses your product the little fellow is sure to follow.

It is the writer's belief that the logical way to handle this specification would be for a disinterested organization to sponsor the work, say the American Concrete Institute, which would appoint a committee of producers and non-producers. The producers would naturally be the concrete stone manufacturer and others associated in the production of the stone, while the non-producers would be architects, engineers, or anyone else who might be interested in the design, manufacture (indirectly, however), and use of this product. The services of stone contractors and general contractors might be used to advantage among the non-producers. The American Society for Testing Materials has used this method for years in writing standard specifications, with remarkable results as you all know. But the writer believes that the committee, if one is appointed, should be sponsored by a disinterested national organization rather than a trade organization, since architects and engineers will have more faith in a publication issued by the American Concrete Institute, the American Society for Testing Materials, the U. S. Bureau of Standards or similar unbiased organizations.

There is another objection that the architect finds with the average concrete stone manufacturer, and that is lack of co-operation. We recently had a job where we wanted to use a buff sandstone. As it was winter and the quarries were closed, we went to a cast stone manufacturer who said he could make it; but the results were awful. Apparently no attempt was made to show the natural veining of the stone, and the finish was like a cement sidewalk. The manufacturer frankly stated, when the samples submitted were criticised, that no one could make what we wanted. Possibly this was his limit, but another manufacturer worked with us and developed a color, veining and texture that was satisfactory. From this one incident, I am convinced that if the makers of concrete stone will exert themselves a

little they can develop a product that can be used to advantage in many places. But it means work, study and co-operation on their part and the development of an educational campaign with the architect.

Still another objection which the architect has to concrete stone is the cost. For example, on a recent project in Detroit calling for approximately 100,000 cu. ft. of stone, an alternate price was asked for concrete stone facings in place of "select buff" limestone, with a "smooth finish free from tool marks," to use the limestone producers' description of their finish. Prices were to be based upon the stone erected in place in the building. The average price submitted for cast stone was $3.18 per cu. ft., while limestone was quoted at $2.60 per cu. ft., a difference of $0.58 per cu. ft, or a matter of some $58,000 for the entire job, which you will all grant is too much. If the prices had been reversed, the concrete stone proposition would have been thoroughly investigated and it would undoubtedly been very favorable to the concrete stone in view of some of the stone jobs that have been built in the last few years.

To sum up the situation, the architect and engineer are both ignorant of what can be done with concrete stone. Much excellent work has been done in this field, and many monumental buildings have been erected. Why cannot the concrete stone men get together and if need be form an association as other lines of business have done, develop a set of specifications, a set of standards, and a code of standard practice so that the architect may know, that if the manufacturer with whom he is dealing or with whom he may wish to transact business is a member of this organization, his product is guaranteed. The association could also act or establish a research bureau, and this would be a place for the exchange of new ideas.

All this is in addition to the specification committee previously mentioned. It is well to remember in this connection, architecture and engineering is dependent on the exchange of ideas and a correlating of the theoretical and practical.

Now let us consider the so-called masonry units, namely, concrete block, tile and brick. In the first place, we should consider the weight of concrete units for the backing of wall facings, for fireproofing structural steel and for use as curtain walls and dividing partitions. It is easily understood that every additional pound used in the construction of walls, partitions, fireproofing, etc., of a building means an increase in the size of the steel members and therefore increasing the size of foundations. One way that this objection can be overcome is by using lighter aggregates without sacrificing strength. One method, that of using washed cinders, is nationally known.

In the second place, the appearance of the average block is very much against it. Let us quote from Hool & Johnson "Handbook of Building Construction," page 988:

> "The so-called rock-face block has done much to injure the standing of the material (concrete block) generally, another outcome of the desire to imitate. Plain rough blocks, made with aggregate as coarse as the limitations of manufacturing operations will permit, make a handsome wall."

Third, some better method should be developed for handling concrete units of all kinds. Usually the units are dumped from a truck or stacked in such a manner that the edges are spauled, chipped and broken. When tile damaged in this way is laid up in the wall it is unsightly, unless very carefully filled with mortar, and is certain to permit moisture to enter.

There is little need to say anything about a standard specification for block, as we have the specification of Committee P-1, of the American Concrete Institute, which can readily be amended as the conditions demand. But a fourth thing that the writer believes is essential is impressing upon all makers that their blocks should be branded for ready identification so that a contractor can be sure of the product he is using, particularly as many product manufacturers sell their output through building supply dealers. Supply dealers have been known to play one maker against another and then indiscriminately mix the blocks of two makers in the same load.

The architect, the engineer or the contractor, in applying specialized knowledge to specific problems, is often confronted by the necessity for immediate and accurate information regarding the new and less academically familiar forms of building material. Therefore the concrete product manufacturers should put before these men the new uses and all the facts regarding the nature, possibilities and correct uses of their products.

To summarize, there is good and bad concrete stone, with the bad predominating, which means that the stone manufacturers must get together and establish standards of material and workmanship, if need be forming a trade association to enforce the standards when once established. Good work can be turned out, as many excellent jobs prove. The concrete masonry units industry has been fairly well organized and is producing a very uniform product, but more study needs to be given to reducing the weight of the products to bring them into more general use. All manufacturers should work in closer harmony with the architects, engineers, contractors and with each other.

Specifications for Concrete Stone.

By C. Van de Bogart.*

The Economy Concrete Company joined the American Concrete Institute for two reasons: First, to gain current knowledge of making concrete in other ways than of the concrete stone industry because we realize that the fundamentals of good concrete are the same no matter where or how made, placed or used, and that these fundamentals are important in making our cast stone. Second, to establish better acquaintance and understanding with other manufacturers of concrete stone through convention contacts and other means possible in this organization. Back of these objects was the conviction that the concrete stone manufacturers who last year marketed products whose estimated value was more than $50,000,000, should have an association all their own. This is with no thought to duplicate the excellent work of this Institute from which the stone manufacturers must benefit, but to accomplish other things in the adjustment of relations in the concrete stone industry and the raising of the standards of that industry in ways which are properly the work of a trade association and not of a technical society.

First of all, it seems, among the needs of the concrete stone industry is a standard specification—a statement of the nature and characteristics of the product we are making. Such a specification should come from the American Concrete Institute through the work of a committee on which both producers and users—manufacturers, engineers, and architects—are represented.

Because the stone manufacturers have, in general, only begun to have a new group consciousness, a feeling that they must work together, there is a little apprehension that they will find it hard to settle upon a specification which can be generally accepted but perhaps they can make some progress among themselves which will make for proper committee procedure, if they thresh out in discussion among themselves some of the preliminary steps. That is why, it seems, an association of stone manufacturers is right now important.

No Institute committee should undertake the specification problem unless there is some hope of general agreement. My purpose here is merely to outline the need, doubtless what is already in many of your minds. If now, out of a discussion we may get no further than to agree to attempt an agreement it will be gratifying progress. It must first be decided that we do, or do not want more unity and group cohesion among manufac-

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turers of concrete stone, and a dependable minimum standard of quality. If we do seriously want these things and recognize their importance, we may all be in better humor to compromise our minor differences for the sake of our agreement in the major object.

As a matter of a trade association does not properly come before this Institute, I will now confine myself to the subject of specifications, although eventually I believe a manufacturers' association could help not only in perfecting them but in maintaining them after they have been adopted. Some manufacturers perhaps view this subject as a personal matter. They may feel that conditions properly vary with locality. However natural this conclusion may seem, it is becoming increasingly apparent that the specification subject is distinctly a group problem and that it must be solved in order to meet the request of architects, and others, for specification data from the American Concrete Institute.

It should not be difficult to convince any concrete stone manufacturer, regardless of the nature of his particular product, that the fundamentals of his good concrete are identical not only with the fundamentals of every other manufacturer of concrete stone, but with the fundamentals of good concrete however made or placed. It is this which establishes our position in this Institute and makes our interests in those fundamentals identical with the interests of all other members, and it also establishes our own special community of interest as manufacturers of concrete stone.

Whatever special operations we may have in proportioning, mixing, conveying, placing, curing or surface treatment, no matter how these vary in detail of conditions, we are all producing something which is subject to like laws of expansion, contraction, to crazing, to the use of color and other admixtures and involving the same rules of physics and of chemistry and subject to the same destructive influences. Obviously, we have a common ground of interest.

It is a well-established fact that a specification in the construction field (or perhaps anywhere else) should be, to do what is expected of it, the joint effort of manufacturer and user.

Otherwise, it doesn't mean anything to architects and engineers who may be expected to specify the material. It must, to command the respect of building authorities, come from an organization which gets its scientific standing from the fact that the maker and the user sit down together. When it comes to a recommended practice as distinguished from a specification, the manufacturer has much more to say. The former says how it shall be made and the latter says how it shall perform and what its characteristics shall be.

The demand for standard specifications most assuredly exists and in searching for particular reasons for their existence several can readily be discovered. It seems therefore particularly fitting to me that we should avail ourselves of the opportunity existing through our connection with this Institute in whose membership we have so many specialists, at least to lay a foundation for a specification which will not only improve our

product but promote a reasonable degree of confidence that our product will be well handled and placed after passing from our own hands to that of the builders who will use it.

A specification for concrete stone must necessarily be divided under two headings: First, the characteristics of the stone itself; second, the result after setting and pointing.

Much faulty practice exists today in manufacturing concrete stone. While a specification will not directly correct poor manufacturing practice it will do so indirectly by creating a standard of quality which must eliminate poor practice. A poor product anywhere lowers the public esteem for the whole class of products to which it belongs.

A makes good stone. *B* makes inferior stone. *B* trades on the reputation of *A*, and *A* loses by the bad reputation of *B*. That is what it means to have an industry without a standard.

Many of us are familiar with the distribution of propaganda by interests with which we are in competition, and their scrutiny has extended completely across and up and down the country. They have rewarded themselves and wrought much damage to the cause of concrete stone through their efforts.

They have found many instances where the composition of concrete stone has been of the most inferior of aggregates with no thought of producing a pleasing color, and a surface finish comparable to the trowelled surface of a sidewalk. Shall such material establish the public's rating of the fine product others are making?

Buildings are found trimmed with stone showing a lamentable lack of reinforcement, with many pieces showing large cracks and superimposed loads grinding away daily at their task of breaking down the stone work.

Other work clearly shows that improper delivery and storage methods at the job sites, together with general careless handling and shipping of material, sometimes cast only a day previous, also exert an injurious effect on the final appearance of the material and in all cases leave glaring and unsightly patches and repairs to further unfavorably influence the use of this product in which we are all so much interested.

These things and other items such as intelligent and carefully prepared setting plans; co-operation on structural details with the builder and architect; facilities for testing, and the careful planning of the delivery procedure, unless ably handled by the manufacturer, also lessen the value of concrete stone as a desirable material to use.

When concrete stone is carefully manufactured, delivered in good order in well-made pieces at the building site, the manufacturers, architects, and owners are not yet assured of a gratifying result. The results of setting and general misuse by mason contractors is the most disappointing feature which I am familiar with, and I am firmly convinced that sooner or later the industry from necessity will be obliged to institute an educational program among the architects and builders to overcome such methods as are now practiced.

It is illogical, but is nevertheless true enough, that if a masonry structure trimmed with cast stone is not watertight, it seems prima facie evidence that the stone is at fault and the manufacturer of it is immediately placed in a defensive position; obliged to expend both much time and money in investigating the condition of the building. Usually he is successful in locating some other cause for the trouble.

It takes generally a very inferior piece of concrete actually to leak water, but the most nearly waterproof material, if not tightly set together at the building, according to the approved practices, will be no better than the mortar which binds it together.

Pointing and protection of the stone are two other important items and too great care cannot be exercised in either case, especially with pointing. Many buildings would appear to far better advantage properly pointed, and not the least important is the final cleaning down of the work which is done best only when done properly.

Why then should we expect to have our work acceptable in the highest degree when the only likelihood of doing so is when we have a perfect combination of architect, builder, manufacturer and owner—combining a knowledge of what to specify—how to manufacture—how to erect—and what to pay for?

This is our only expectation of improvement and it cannot be hoped for until we ourselves consider carefully how best to overcome the conditions which defeat the purpose.

Some apprehension may exist as to our ability to adopt a specification which would apply to wet-cast and dry-tamped stone, but my judgment leads me to believe that it can be accomplished. At least in the effort, we would be parted from nothing except a little time and a few opinions.

The many considerations already stated in conjunction with the amount of capital invested, the years of expensive experience, and the constant study to improve our product, make it almost a necessity that we take the next steps—a concerted movement, with the greater force of united effort over many unrelated individual efforts on the part of all those anxious for progress, advancement and proper development of the industry.

In my opinion, and I submit it to you for your consideration, there are two things to be done—adopt standard specifications through the American Concrete Institute and for work preliminary to that, and to supplement and to thoroughly establish those standards where adopted, the organization of the concrete stone producers, into a group association in which and through which we may undertake to correct the haphazard conditions of our industry. Let us not fail to balance the advantages of a standard specification against any possible shortcomings and if there is sufficient evidence in favor, set up a preliminary organization to formulate plans and method of procedure with the view of improving our position and our product.

Need of National Certification Plan for Cast Stone Industry.

By M. A. Arnold.*

In this paper I shall endeavor to discuss a problem which has confronted the cast stone industry for a number of years, namely, standard specification.

With a few exceptions, practically all of the major industries have adopted some method of standardization, realizing the industrial and economic significance of a standard specification. Equally as large industries, with as many various methods of manufacture have solved their problem of standardization by means of a process of elimination or simplification.

In the past few years the writer has had the opportunity to discuss with many of the leading cast stone manufacturers the subject of standardization and found that the majority of producers would welcome a properly prepared specification which would clearly describe the qualities and characteristics of the finished stone without attempting to set up a recommended practice of manufacture.

Many of the producers seem to be laboring under the impression that they are using their own individual secrets and formulas in the manufacture of cast stone, when, as a matter of fact, the fundamentals of concrete apply to all methods of manufacturing cast stone, and with a reasonable amount of study, almost any manufacturer can duplicate his competitors' colors and textures. If we are to be progressive, if we are to succeed in our efforts toward standardization, we should welcome our fellow manufacturers and freely exchange ideas, establish closer co-operation among ourselves and consider the common good of the industry.

In a few years the cast stone industry has grown from a small beginning as a side line in a backyard concrete block plant to an industry producing millions of dollars worth of stone annually, this being used in some of the country's largest projects. Is it not appalling that such a gigantic industry could be satisfied with the present hit-or-miss methods of manufacturing? The time is fully ripe for all interested manufacturers to make a start in the proper direction.

Architects and engineers attempting to specify the product have quite a number of names to select from, such as art stone, cut art stone, artificial stone, cast stone, cut cast stone, composite stone, concrete stone, pre-

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cast stone, cement stone, imitation stone and others. This often results in considerable confusion. The first preliminary step to be taken should be the establishment of a proper "term" relating to the product that would describe and distinctly classify the type of stone. Nothing is more disconcerting in business transactions than the loose and incorrect use of terms upon whose interpretation the meaning and intent of a contract may turn, and much difficulty will be avoided if such terms are clearly defined. While personally, I believe that the term "concrete stone" is the correct term to use, I fear that the consumer is not sufficiently informed to appreciate the correctness of the term and its use would have a tendency to place our product on a level with rough unattractive concrete. The term "cast stone" would be more desirable, since it is more widely used and clearly describes our methods of manufacturing. Casting according to Webster means "the act or process of founding or moulding, or the process of taking impressions."

The next preliminary step should be the proper classification of the various methods that are now used throughout the country. To begin with let us analyze the fundamental principals of both the dry-tamp and wet-cast methods. Is it not safe to assume that both types of stone can be classified as "concrete" which is a mixture of cement, water and inert materials and irrespective of the consistency of the mixture used, is being molded either in sand, plaster, wood or gelatine?

Assuming the term "cast stone" be used, the next step would be to separate the two methods into two distinct classifications. The dividing line is reached when the qualities and other characteristics of the stone, such as compression and absorption requirements and surface treatment are so considered. Surface treatment of the stone appears to be one of the most important factors and should receive careful attention, as this particular item will greatly help us to arrive at the desired classification, namely "cast stone" and "cut cast stone."

At this time I should like to offer a few of the most important items which will require considerable study for both specifications.

CAST STONE.

(a) *Surface Finish:*

All plain surfaces, mouldings, ornamental work, etc., shall either be "unfinished," "trowelled," "sprayed" or "acid washed."

(b) *Materials:*

Materials shall be such as will meet standard requirements as well as colors and textures desired.

(c) *Compression Requirements:*

(To be determined by a series of tests.)

(d) *Absorption Requirements:*

(To be determined by a series of tests.)

CUT CAST STONE.

(a) *Surface Finish:*

All plain surfaces, mouldings, etc., shall be "re-cut" by hand or machinery. All ornamental work shall be re-carved to insure sharpness of detail.

(b) *Materials:*

Shall be such as will meet standard requirements as well as colors and textures desired.

(c) *Compression Requirements:*

(To be determined by a series of tests.)

(d) *Absorption Requirements:*

(To be determined by a series of tests.)

This is only an attempt to illustrate to you the possibility of a national certification plan, and considerable research work and tests determining compression and absorption requirements for both types of stone will be necessary before a specification can be written. When a specification is being prepared this problem should be placed into the hands of a committee appointed by the American Concrete Institute and consist of three groups—producers, consumers, and independent experts such as consulting engineers, etc.

Both producers and consumers undoubtedly will be benefited by the adoption of a standard specification. It will result in more truly competitive bidding on the same quality of stone, greater uniformity, economy and simplification in methods of manufacture. The importance of standardization in relation to the future development in the industry can hardly be over-estimated, and the adoption of a national certification plan should receive the whole-hearted support of every progressive manufacturer.

Discussion.—National Certification Plan.

Frederic M. Emerson (*By letter*).—Being especially interested in the subject of standard specifications for concrete stone, I would like to make a suggestion which I hope may help lead to some definite standard. Mr. Emerson.

We will assume that the architect of the prospective structure has formed some mental vision of what he proposes to erect and, therefore, has formed an opinion of the character of stone he wishes to adopt. Having definitely framed this opinion, he should request one or more of such concrete stone manufacturers as he deems of sufficient experience, and in other respects acceptable to him, to submit samples, describing to them the particular kind of concrete stone he wishes to adopt for the particular structure in hand.

The architect then selects one or more samples, which we will assume are satisfactory to him, both as regards physical characteristics and architectural features. He then identifies them by labelling or tagging, "Standard Sample of Concrete Stone for X Building."

The matter of the specification now becomes most simple and should be stated in substance as follows: "Concrete stone to be incorporated in X building shall conform *in every respect* to standard sample on exhibit and identified in architect's office," or, if the architect wishes to make the specification even more rigid, he may use the following specification: "Sub-contractor on cast stone must be able to show .... buildings, ... years old, containing concrete stone of his manufacture which in the opinion of the architect is of such quality and in such condition as will sufficiently establish said sub-contractor's technical and artistic ability in the art."

Assuming that the architect has selected a high class product as a standard sample, the above specification, naturally, must insure only a high class result, provided, of course, that Mr. Architect insists on the requirements of his specification.

The writer believes that the above suggestion may be helpful to architects in obtaining exactly what they will require. At the same time it should greatly simplify the present condition, when so many architects are groping and trying to explain in writing what they are trying to obtain, some even trying to describe in detail what they believe should be the fine technique of manufacture to produce what they have in mind.

The subjects of reinforcement, handling, setting, pointing, cleaning, etc., can more easily be standardized but the writer believes that these other subjects do not offer the same complications as the subject touched

upon above. The proportions of mixes, manner of casting, etc., cannot be standardized, as the architectural requirements in each individual case, naturally, will call for different mixes and manner of finishing.

If the concrete stone manufacturer has been in the business a sufficient period and has produced enough stone to establish the fact that he can and does produce only a high class product, and has established a reputation of a high order thereby, the architect should assume that he will continue to do so. Therefore, it does not seem necessary that the architect should feel it necessary to try to describe in his specifications all of the techniques of manufacture, in order that he may obtain such a stone as he may have in mind.

Pacific Stone. A Dry Tamped Product.

By Gilbert E. Tucker.*

The Pacific Stone Company of Seattle, Washington, is the largest firm producing high grade cast stone in the northwest today. Our plant now covers an entire city block and facilities are such that orders of any magnitude are produced and delivered with a minimum of delay.

I do not intend to dwell upon the necessity of a well-organized drafting room, of maintaining an expert modeling staff, or the methods of economically and scientifically making wood and plaster patterns, but the actual fabrication of what we have found to be a successful dry tamped cast stone.

While the company's entire present management for years were men who had formerly been connected with widely known cast stone firms manufacturing wet cast stone of the very best quality, we found conditions to be such that it was necessary to develop dry tamp stone or withdraw from the market. I mention this to bring out the fact that here is a firm thoroughly versed in the advantages and disadvantages of wet cast stone, who feel that through exhaustive and intensive research work on dry tamp products they are now producing a stone equal to the best.

In our casting shop the front of the building is designed to receive the various sizings of aggregates and cement in shutes which carry the materials into large bins located behind the backing and facing mixers. The aisle between the mixers and the hammermen's runways are used to convey the various batches into the individual partitioned boxes holding both facing and backing materials. As nearly all our jobs call for at least three shades of color, a hammerman after ramming the first stone from material taken out of his own box uses the hammerman's box to the right of him for the second stone and to the left for the third stone. The runways running the whole length of the shop are constructed of two greased 2 x 6 in. planks placed 26 in. apart and properly bolted and braced 20 in. off the ground. Directly at the end of the runways are the steam kilns. After the stone is properly cured the back door of the steam kiln is opened and we find the stone in the shipping yard ready for acid cutting or tooling as the case may be.

We seldom vary from a 1: 2½ facing mix composed of different sizes of hard aggregates carefully graded and ranging from ¼ in. to almost dust. The proportions of fines are naturally held down to a minimum. Mineral colors are used only to tone down the white cement in forming

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a pleasing background for the many highly colored aggregates that we are fortunate in having a choice of on the Pacific coast.

The backing material is a 1: 2: 3½ mix of clean, carefully graded washed sand, roofing gravel and a gray waterproof cement. Seattle, with its climate of rain, sunshine and sometimes snow within a twenty-four hour period, and continuing in this manner for the whole of the winter months, while still a charmed land to live in, was the despair of both clay brick and cast stone manufacturers in showing up a beautiful coat of efflorescence. We have been particularly fortunate in this regard and I firmly believe it is because of the use of 4 lb. of barium carbonate per sack of cement in our backing material. While I do not intend to dwell upon the chemical analysis of the use of barium carbonate, I feel it would be well for those experiencing difficulty in combating efflorescence to make experiments along these lines.

In determining a water ratio for dry tamp stone, we must consider that as our plaster or wood moulds must be stripped down immediately after ramming, the stone must be dry enough not to warp or fall apart. The practical method is to keep the mix so that it easily balls up in your hand and yet shows a distinct water web when pressed against a trowel. Even during the initial set of the stone, we throw on a fog spray. In short, we are aiming at a semi-wet method of casting stone, and exceptional care and experience is needed at this point of fabrication.

Our hammermen are equipped with model No. 4-B Chicago Pneumatic hammers and Ingersoll-Rand hammers of like size. They have a stroke of 5 in., and at the maintained pressure of 100 lb. they strike from 1,000 to 1,200 blows per minute. The usual butt of these hammers is 2½ in. in diameter, but we keep various sizes in stock for special stones. As you can imagine, our patterns are made exceptionally strong to withstand this terrific pressure. The facing material is first thrown sharply into the bottom of the pattern and packed in all corners by hand, care being taken that a full 2-in. thickness is maintained; then 2 in. more of backing is added and the first machine ramming takes place. Approximately 4 in. of backing is placed for each ramming thereafter until the required depth is reached. In order to eliminate any lines of demarcation between ramming courses the stone must be sharply scored with a trowel. The bottom pallet is then clamped on, the pattern box turned over and pushed down the greased runway and stripped. As the stone is now exposed with facing material up, the hammerman can do any slicking necessary while the "turnover man" sets up the pattern box for the next stone. Directly overhead are lines of fog spray sprinklers which are turned on at various intervals.

As soon as the stones are in a condition to handle safely, they are conveyed into the steam kilns by lift trucks. A line of ¾-in. steam pipe, perforated every 2 in., is run into wooden troughs which are placed 8 in. off the floor. These water-filled troughs are made of white pine approximately 6 in. in width and 6 in. in depth. The roof is V-shaped of cor-

rugated galvanized iron. At 8 lb. pressure we are able to maintain a constant heat of 100 deg. F. This steam vapor rising, hitting the roof and condensing forms what I should consider a water-vapor cure. The method of live steam curing implies to the architect something that is drastic and accelerated and therefore objectionable. We feel that he is right and therefore are attempting a water-vapor cure. While three days in the kiln is all that is necessary for acid cut stone, we obtain better results at 10 days for stone that is to be tooled.

At least 80 per cent of our stone work is acid cut, by etching the surface with muriatic acid and water. The condition of the concrete is the main factor governing the proportions of water and acid, but in general a 1 : 3 proportion will give the desired texture. Even the job of acid cutting requires care and experience or the stones will be ruined. We have some fine examples of carving and tooled work in the University of Washington buildings.

The qualities of high compression, low absorption and modulus of rupture in a transverse test I believe to be the final requisite of a high-grade cast stone. In going through files and tests made by various testing laboratories on our material during the last three years I am unable to find any single test where the compression is lower than 6,000 lb. per sq. in. or the absorption higher than 4 per cent. In obtaining our certificate of quality in the Northwest Concrete Products' Association we had to submit our stone to a series of tests. Allow me to quote from the resultant report: "An average of 7,500 lb. per sq. in. crushing strength was obtained on a set of 3 cylinders made from the regular backing mix. An average of 6,500 lb. and 9,250 lb. per sq. in. was obtained from the two different types of facing materials." The material for making the cylinders was taken from the regular batch and without advance information to the workman. All cylinders were made as near the shop practice as possible and cured with the regular run of cast stone. They were placed in the steam kiln for 10 days and for the balance of 28 days received ordinary air curing. The cylinders were then broken at the structural materials laboratory of the University of Washington under the direction of Professor Ira L. Collier and F. R. Zaugg of the Seattle Portland Cement Association. The average absorption of all cylinders was 1½ per cent. These tests were obtained from the batches on two jobs that were being made at the time.

Our company was recently asked to figure a new 27-story building with the intention of facing the entire structure with our material. The architect had never used cast stone except on a few small jobs and therefore obtained the services of the leading testing laboratory in the northwest to make a complete and extensive survey of our material. Their final report covers five pages, so I will just give results obtained. Backing material was 7,627 lb. per sq. in. Facing material was 6,770 lb. per sq. in. Stones with no surface waterproofed showed an average absorption of 1.16 per cent. Stones that were surface waterproofed showed an absorption of 0.8

per cent. The modulus of rupture in a transverse test with facing material in tension showed an average of 967 lb. and with backing in tension 897 lb. In no case was there a sign of cleavage between backing and facing material. Tests were made to produce stain in contact with different metals without unfavorable results.

Unquestionably the manufacturers of cast stone, whether it be dry-tamp or wet cast are being seriously hampered in their progress by firms caring little about quality. It would therefore appear that regardless of our private opinions, we should not question each other's methods of production but band together in an effort to standardize a set of specifications embracing both methods with but one point in view—to manufacture and produce a high quality of cast stone.

Discussion—Pacific Stone.

Austin Crabbs.—I do not think too much stress can be laid on the value of adopting a definite name for concrete stone. In choosing that name, the psychology of the name itself in furthering sales should be taken into consideration. We have been making stone on a rather small scale, and about a year ago we started calling our product architectural stone. This is a somewhat similar name to art stone and other terms used by some other manufacturers, but we find that we get more attention when we talk about architectural stone than about cast stone or something of that kind. Our experience has convinced me that the method of approach has a great deal to do with the eventual success of a name. Mr. Crabbs.

While in many cases it is more satisfactory to say architectural stone, in others we must define that a little more, and we say architectural cement stone or concrete stone. I feel that the expression synthetic stone is rather in disrepute. As brought out by Mr. Arnold's paper, the architects and engineers as a whole are not particularly interested in how the stone is made, and the more that question is stirred up, the more confusion will arise. I think it is better that some specification should be used giving the result, and not dealing with the method of obtaining that result.

Earnest Kremers.—Why use the word stone? It seems to me that stone in the English language means a natural product. It would be much more definite and honest to call this product cast concrete. Mr. Kremers.

C. A. Bullen.—I believe there is only one term that should be used if we are to be properly understood, and that is cast stone. In making various kinds of stone, stone is used as an aggregate. In many parts of the country an imitation marble, limestone, or granite is being made and in each instance they use natural granite, limestone and marble. To get down to a definite basis, this concrete product should be called cast stone whether it is synthetic or otherwise. Mr. Bullen.

J. W. Mueller.—I speak as a consumer. I believe that the consumers would be much better satisfied if the Institute were closely identified with standard production practices as well as with details of design and methods of selling. Mr. Mueller.

L. A. Dalco.—I would like to move that the Board of Directors appoint a committee on the matter. Mr. Dalco.

R. W. Levinsohn.—From the long experience that most of the architects in the country have had in writing cast stone in specifications, I believe it would be very hard now to educate some of them to use another term. Mr. Levinsohn.

Mr. Bragger. E. Y. BRAGGER.—For many years this problem has been a serious one for architects. They do not know what to specify and have been governed largely by appearance and more especially by price. I feel that we need closer co-operation among the manufacturers. There is hardly a cast stone job but what could have been improved in some little details. If it is beyond the power of that manufacturer to solve any particular problem, let him be sufficiently close to his neighbor to call him in and find out what the trouble is. Other industries are doing it and are solving their problems. I strongly recommend that the gentlemen interested in that problem do not stand on high pedestals but give special attention to the younger members of the family and do everything they can to encourage a healthy growth of that which will be, at some time, a good growing concern.

Mr. Arnold. M. A. ARNOLD.—If an Association were to be formed and some specification were to be drawn, my firm stands ready to adopt the name concrete stone immediately. For as mentioned in the paper by Mr. Eyrick, every architect is against an imitation of any kind of material. If we adopt the name concrete stone, we are getting into our own field where we belong.

Many cast-stone manufacturers are having their troubles about price. They cannot compete with natural limestone, and the trouble is that they are in the business of imitating natural limestone. In our part of the country we make an effort to create an individual texture and color for concrete stone. If the architect likes it, he will specify and use it in spite of price.

Mr. Van de Bogart. C. VAN DE BOGART.—I do not believe that the Institute should attempt to hold jurisdiction over what this material should be called. I believe we should decide on our motion on a committee. In due time it will be appointed, and then it will be up to the cast-stone manufacturers themselves to decide on what they want to call their own material. (The motion was unanimously carried.)

Formulating Portland Cement Stucco.

By William S. Steele.*

Formulating portland cement stucco is my subject, and I propose to treat it from a commercial, practical and semi-technical standpoint, rather than from the graph, chart and statistics of the laboratory.

Factory pre-mixed stucco, in colors, is increasing in popularity, and commands a good price in the current market. This demand is a lure to potential manufacturers who see in the fall of magnesite products a bonanza in the portland cement field.

The decline in plastic magnesia products, and the unfavorable showing made by magnesia stucco can be attributed largely to lack of foresight and unified control of its manufacture by the raw material producers. The embryo manufacturer was led to believe that magnesite was a fool proof mineral and that its conversion into a durable stucco was only a simple mixing process. Experience was unnecessary, and profits were so large that a minimum of capital was needed. Countless instances could be cited where a raw material broker handed out rule-of-thumb formula and extended huge credits to inexperienced men to promote the sale of their products.

A small percentage of these manufacturers made a serious study of their field and put out products that were properly proportioned, and tried to instruct the plastic trades in honest stucco application. This class was the first to see the handwriting on the wall, and to accept in the increasing popularity of colored materials, (unattainable with oxychlorides) an opportunity to devote their plants and organizations to the mixing of portland cement stucco. Because of years of experience in the blending of stucco and first hand knowledge of its application, these manufacturers are a valuable addition to the portland cement field; and as a member of this group I plead for an open arm reception of the responsible ex-magnesite stucco makers.

It is not possible nor practical to lay down an iron-clad set of rules for the guidance of all stucco manufacturers. Prohibitive freight rates, for example, would reject for a mid-western plant the aggregate used on either the eastern or western coast. Climatic conditions in the north and on the Atlantic seaboard compel certain precautions and the use of ingredients that are unnecessary in California and Florida. Still it is vitally important that high standards be raised and insisted upon to govern the manufacture and distribution of portland cement stucco, and to insure

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for it the place which it merits in the field of building materials. Consideration of stucco bases should be thorough, and the findings should be broadcast for or against each stucco backing offered for sale.

The portland cement industry is in a position to control and recommend practices where the base is concrete or masonry. The range is so varied over frame construction that the observations of each stucco manufacturer should be placed in centralized hands to assist them in passing judgment on patent bases for the benefit of us all. Results desired, rather than proportions and ratios of ingredients, must form the basis for manufacturing specifications. Past experience must dictate a given minimum of tensile strength required for acceptable stucco at the end of 7, 28 and 90 days, respectively. A substantial margin of safety should be added to these figures and the totals set as minimum standards for approved materials. Compressive strength and abrasive resistance are unimportant factors in stucco, since stucco is not called upon to endure either compression or abrasion, under normal conditions.

Plasticity is a factor of importance commercially, but it is a quality that adds neither strength, permanence, nor resistance to the material in itself, and can safely be determined by each manufacturer, where added plasticity is not obtained at the expense of more important standards of safety.

Water-repellance, absorption, time of set, density, and shrinkage are so closely related that a given coefficient of expansion and a predetermined density will control these other properties. Injurious admixtures and fillers functioning as accelerators, waterproofing agents, fatteners or void fillers, in most cases will cause undue shrinkage and cracking, and automatically eliminate their use. Here again, climatic conditions prevent a nation-wide ban or approval of certain items. Lime, for example, should not be used where the stucco wall is exposed to sea-air or alkaline water; yet its use might be an advantage under different atmospheric conditions.

Coloring pigments must be made the subject of intensive research. Should the color proportion be deducted from the volume of cement or aggregate? Should earthen colors be used? Are chemical precipitates permanent? What effect does the iron oxide content of a given pigment have on the perfect hydration of the wall, and permanence of the selected shade? These questions must be answered and coloring agents selected that are non-injurious, fast and uniform.

Fire-resistance is an important factor in stucco. Avoid the use of fillers and pigments that decompose readily under heat, for decomposition of particles in the material leave a pitted and porous surface. A well-balanced material will also provide strong resistance, to lateral strain. It would be well to include such a feature in our specifications.

Since coverage and guarantee of spreading capacity are important sales arguments, there may be a tendency to provide a bulky package, insuring increased yardage at the expense of vital factors enumerated above. For this reason this practice should be checked through a standardized

"weight-by-volume" specification, although a material measuring up to these standards could not easily carry a harmful proportion of fluffy fillers.

I believe in a uniform specification for the portland cement stucco industry embodying a thoroughly tested, predetermined figure covering each of these properties, and I urge that a separate unit be formed of stucco manufacturers closely allied to and having the whole-hearted support of the Portland Cement Association and the American Concrete Institute, with sufficient prestige and authority to approve materials that are worthwhile, and either to improve or condemn shoddy products.

Since the stucco scratch and brown coats are hidden and covered by the colored finish coat, the largest proportion of these materials will always be manufactured by the mason at the mortar box. For this reason, simple and complete instructions for mixing these coats should be broadcast through the plastic trades, educating the mechanic in correct procedure. Teach him the purpose of each coat; how to kill suction in his base; what to avoid in selecting his aggregates; the correct proportions of cement to sand; whether or not to waterproof; and how long to wait between coats.

The base coat must set rapidly enough to prevent sagging. Make it plastic enough to stay where it is placed, and strong enough to form rigid keys and to provide a permanent and rugged foundation for the succeeding coats. The brown coat shall have strength equal to the scratch coat. Additional plasticity and density is desirable in this coat for ease in spreading over an absorptive base, and to check excessive suction in finish coat application. Instruct the mechanic how to obtain these qualities and how to judge his results, and the stucco manufacturer gains a valuable ally and an insured base for his finish coat.

Given a definite set of finished wall specifications as his goal, the premixed stucco manufacturer must conduct his preliminary research work with the sole purpose of reaching these standards. The products must attain a minimum tensile strength at stated periods. Elementary experiments in the ratio of cement to aggregate will determine the proportions required to attain this strength. Available aggregate supplies should be analyzed and tested, for a balanced ratio of fine, medium and coarse aggregate will control, to a large extent, the coefficient of expansion as well as the density of the product. Determine the percentage of voids in the aggregate and use that percentage as a starting point in determining the ratio of cement to sand. Then increase or decrease in quantity as circumstances warrant.

Now, with a satisfactory blend of sand and cement in the correct ratio, the product may meet all requirements of strength, shrinkage and density, but it is not commercially salable. Cater next to your customer. Plasticity can be gained through many channels. Test them all, and reject or accept them according to the results obtained until your blend still measures up to the standards set, and is at the same time elastic and easy to work.

Is your product now safely water repellant? Portland cement stucco should not be 100 per cent waterproof, for such a mix will check complete hydration. Yet it must be sufficiently water repellant to protect its base. Integral waterproofings may or may not be necessary in your formula, as your tests will show.

Color choice and selection of a standard color range are highly important. Standardize on as few colors as possible to meet the demands of your clientele, and devote all of your efforts to the perfecting and improving of those shades.

This brief outline is, in our opinion, an essential course to pursue in building commercial formulas for pre-mixed stucco. Localities and climatic conditions will determine to a large extent the course of procedure in experimentation, but the fundamental principles are the same whether the town be New Orleans, Minneapolis or Boston. Poorly mixed stucco and faulty application must be avoided if our industry is to progress. Education of the plastic trades in application, and a centrally controlled unselfish manufacturer's association with power to enforce a high standard of manufacture through approval or threat of boycott is, in our opinion, the combination to assure our growth and the increasing approval of our products.

Discussion.—Portland Cement Stucco.

C. G. Walker.—In his comments Mr. Steele has given the viewpoint of a manufacturer of prepared portland cement stucco. I should like to briefly discuss the matter from the viewpoint of those who are charged with the improvement and extension of the use of stucco as one form of concrete—the Portland Cement Association. **Mr. Walker.**

Several years ago the American Concrete Institute established standard specifications on the preparation of portland cement stucco. That was before the time of factory-mixed portland cement stucco and before the introduction of modern textures. While mortar prepared in accordance with the A. C. I. specifications is satisfactory for base coats and for rough cast, spatter dash and perhaps for sand float finishes, it is not possible to use such mortar in putting on modern textures.

Any manufacturer who has looked for published information that might help him in solving the problems of preparing stucco that could be textured with a trowel has found a discouraging lack of helpful data. There is very little information available on any sort of cement mortar, to say nothing of the special kind of mortar that is necessary for modern texture work. Years of study and research have produced a vast amount of data on concrete mixtures and although the principles of good concrete and mortar are essentially the same, refinements in the handling and placing of cement mortars require that the mortar have properties entirely unnecessary in concrete. Such then has been the situation in which prospective manufacturers of portland cement stucco have found themselves.

Coincident with the increasing use of portland cement stucco so many inquiries have come to us from various sources for information on the preparation of stucco, on the effects of various admixtures in stucco and on the quality of factory-mixed stucco that we felt it necessary to obtain for our own benefit definite information as to the composition and quality of portland cement stucco on the market.

At random, samples of factory mixed stucco were picked up in different parts of the country and sent to our chemical laboratory. Three stuccoes of different colors from each of fourteen representative manufacturers were carefully analyzed. It is a current belief that such a mixture as factory-prepared stucco can not be successfully separated into its component parts. While we cannot expect such results to be exactly accurate, we did find by checking them through different methods that they were accurate within a small range.

Analyses showed that there is a great diversity in the composition of different brands of pre-mixed stucco. Proportions varied widely but most of the mixes ranged between 1: 2.5 and 1: 3.5 by weight. Aggregate was generally a combination of crushed marble and silica sand but in some cases crushed marble and in other cases silica sand were used alone.

In the matter of admixtures, we found lime, asbestos and marble dust most frequently used. It was surprising and even alarming to discover in some of these portland cement stuccoes additions of as much as 50 per cent of hydrated lime by weight of cement, which is approximately 100 per cent by volume. Additions of 70 per cent hydrated lime by volume of cement were characteristic of many of the stuccoes. Marble dust was present in some cases in amounts varying from 5 per cent to 100 per cent by weight of cement. In other cases both marble dust and hydrated lime were present in large quantities.

Upon conclusion of the analyses we decided to make standard tests on the compressive and tensile strength and on the absorption of some factory-prepared stuccoes. Specimens were made from only five brands for test at 7 and 28 days of age. For comparison, specimens were also made from a mortar prepared in the laboratory in accordance with American Concrete Institute specifications, using O-8 Elgin sand. All specimens were cured for 5 days under damp sacks and thereafter in the air of the laboratory until time of test. Such curing, as you know, does not give a true cement product a fair chance. However, specimens made from standard mortar were about three times as strong as those made from the weakest factory-prepared stucco, and the strongest factory-prepared stucco was more than twice as strong as the weakest. These relations held for both compressive and tensile strength. The absorption of specimens made from factory-mixed materials was about three times as great as that of the standard mortar after 5 minutes immersion, twice as great after 30 minutes immersion, and 1½ times as great after 24 hours immersion.

In connection with the chemical analyses and physical tests already made on prepared stucco, sieve analyses were made on 10 different brands in order to study the type of aggregate that was being used. The extreme variations found in the grading of aggregate used in these 10 samples indicated that most manufacturers were not paying sufficient attention to this phase of stucco preparation. In view of the very poor grading of aggregate in some of the stuccoes, it became evident why large additions of plasticizing agents were necessary to make the stucco workable and commercially salable.

A study of the effect of size and grading of aggregate on the workability of stucco was begun on samples prepared in the laboratory. Sand was separated into 5 different sizes and recombined to give the grading desired. About 25 mixtures were tried out, varying the size and grading of sand and proportions of the mix. The stucco mixtures ranged between one made in accordance with A. C. I. specifications and a 1 : 2 mix (by volume) using sand all of which passed the 28-mesh sieve and 18 per cent of which passed the 100-mesh sieve. Tests on workability were made by an experienced plasterer who mixed the mortar and then applied it to a panel previously browned in. From the standpoint of plasticity none of these mortars except the ones using fine sand or a rich mix were at all satisfactory for texture work. Time did not permit a more extensive

study but the experiments that we did make convinced us of the important bearing that grading of aggregate has on the workability of stucco.

Recently there has been considerable discussion as to the necessity of high strength and low absorption in finish coat stucco. Aside from the fact that high strength usually accompanies high density, the latter quality of stucco would seem to be more important for numerous reasons. Absorption of water by the finish coat increases greatly the disruptive action of frost. Repeated absorption and evaporation of water from the stucco is attended by volume changes which have a disintegrating and loosening effect on the materials. The wash of rain over the face of a wall removes bit by bit the particles which have been loosened by volume changes or by frost action. Passage of water in and out of the finish coat increases the chances of unsightly efflorescence. A high degree of absorption may be an indication of a condition which allows the slow leaching out of even the relatively insoluble materials of which stucco is made. An inspection of masonry walls will reveal the effect of weathering on mortars, especially near the top and near the grade line where exposure is always most severe. In view, therefore, of all the attacks that finish coat stucco has to withstand, it seems evident that strength and density are necessary.

The installation of a small freezing room in our laboratory in December has given us an opportunity to conduct some freezing and thawing tests on stucco to determine whether there is any relation between its strength and density and its resistance to weathering. A small preliminary group of tests have been started on 8 types of stucco mixtures ranging from straight lime stucco to portland cement stucco with the permissible 10 per cent of hydrated lime by weight of cement. Two mixes were included using two admixtures other than lime and one mix using very fine sand. Specially graded white silica sand was used in all cases, graded as follows: 18 per cent coarser than 28-mesh sieve, 59 per cent coarser than 48-mesh sieve, 85 per cent coarser than 100-mesh sieve and 97 per cent coarser than 200-mesh sieve. Stucco was applied to panels 12 in. square and allowed to cure 5 weeks before being subjected to freezing tests. The test procedure is as follows: Panels are wet by placing them face down in about a ¼ in. of water for about an hour, after which they are placed in the freezing room until they become frozen. They are thawed by again placing them face down in water and the freezing is repeated. While these tests have not been carried on long enough for us to obtain any conclusive results, there is evidence of considerable disintegration of the straight lime stucco. The procedure has been criticized as not being a real freezing test since the panels lose considerable water by evaporation before they become entirely frozen.

Specimens from the 8 types of mortar were made for compression, tension and absorption tests. Volume change studies were also made on thin slabs of the same material. Careful notes were made on the plasticity and other characteristics of the mortars. It is expected that the results of these tests will be useful in outlining a more comprehensive series.

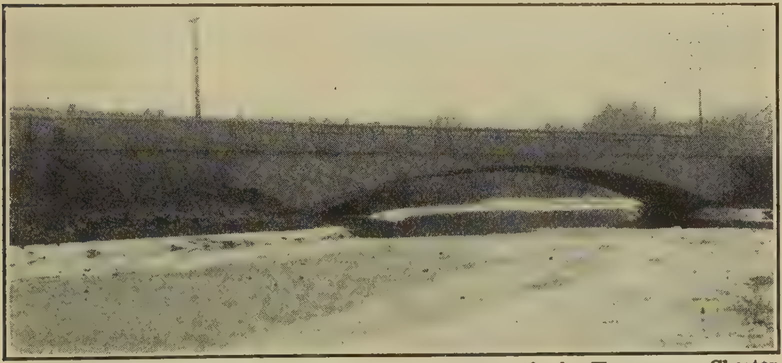
We wish to make clear that our studies and tests have been of a very preliminary nature and that we have reached no conclusions except that further systematic study and experimentation are desirable. We are convinced that the size and grading of aggregate have an important bearing on the strength, density, shrinkage and workability of stucco. While we ourselves firmly believe that there is a direct relation between strength and density of stucco and its resistance to weathering, we will be glad to have this question settled by suitable tests, realizing at the same time that in laboratory practice it will be difficult to introduce and control all the factors that enter into weathering.

We understand that some changes are contemplated in the present A. C. I. specifications on the preparation of stucco. We believe the specifications should be changed in a manner which recognizes the demand for stucco that can be textured. Whatever the changes may be, we sincerely hope that they will include such specifications and tests as will closely regulate and protect the quality of finish coat portland cement stucco.

The Design and Construction of a Skew Arch.

By S. C. Hollister.*

Introductory.—This paper deals with the solution of a skew arch problem, in the building of a bridge in Chester, Pennsylvania, to carry West Ninth St. over the Chester River. The arch has a span in the direction of the roadway of 160 ft., with a theoretical rise of 12.62 ft. The width of the bridge is 60 ft., and the angle of skew is over 42 deg.; hence the proportions are such as to place no part of either abutment face perpendicularly opposite the other. The bridge was designed during the first half of 1925, and construction was started in October of the same year. It was opened to service in October, 1926.



Photographs by Havercamp, Chester.

FIG. 1.—GENERAL VIEW LOOKING DOWNSTREAM.

The paper will treat in sequence a discussion of skew arch action; the design of Ninth St. Bridge; the construction of the bridge; and certain strain-gage measurements made upon the span after completion. The writer has kept free from long mathematical discourse, in order that conclusions regarding each step requiring decision could be set forth with clarity.

Mechanical Behavior of Skew Arch.—A skew arch is different from a right arch in that the abutment face is not perpendicular to the spandrel

* Consulting engineer, Philadelphia.

wall and the abutments are not wholly opposite one another. Stresses due to the loadings will follow paths of greatest stiffness. Thus, in a skew arch the paths of stress undertake the shortest route to the abutment—they tend toward a direction normal to the abutment face. Resultant stresses therefore are in planes not parallel to customary planes of loading, hence distortion takes place transversely as well as vertically and longitudinally.

Consider the plan of a skew arch shown in Fig. 2 (a). The roadway runs in the direction *BD*. A load at the center of the span would set up stresses tending to reach the abutment by the shortest route. Thus the load would tend to be transferred through the arch in a direction perpendicular to the abutments *AB* and *CD*. The load on the abutment *AB* would not be uniform over the length *AB* but would be greater at *B*. This would tend to thrust the end of the abutment *B* a greater distance from the center of the span than it would *A* and similarly would depress *B*

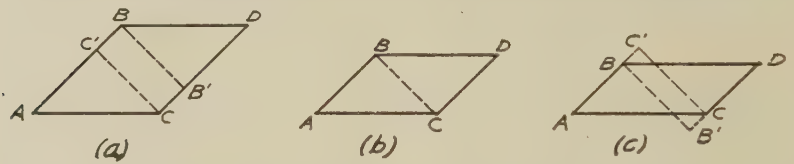


FIG. 2.—TYPES OF SKEW ARCH.

a greater amount than *A*. The originally unloaded abutment line *AB* would therefore be rotated about some horizontal axis normal to the abutment, also about a vertical axis through the abutment. These two distortions are in addition to the only rotation usually considered in a right arch, namely, that caused by moments about a horizontal axis parallel to the abutment face.

Distortions on the abutment *CD* due to this central load are similar to those on the abutment *AB* except that distortions of similar kind are found in one abutment on the opposite side of the center line of the roadway from those on the other abutment. It is plain, therefore, that the span is subject to torsional forces acting about a longitudinal axis as well as about a vertical axis. These are in addition to the customarily considered bending moments about a horizontal transverse axis.

A perpendicular to the abutment *AB* drawn through the end *B* cuts the abutment line *CD* at the point *B′* which lies between *C* and *D*. Similarly, a perpendicular drawn from *CD* at *C* cuts the abutment *AB* at *C′*. The portion of the structure *C′B B′C* is in fact a right arch and will tend to act as such. If, by altering the dimensions or shape of the span one can increase the proportion which *C′B* is of *AB*, one will be approaching resulting stress conditions similar to those in a right-arch span. If a conduit or tunnel lining were under consideration where *AB* is very long with respect to *AC* then the portion *C′B B′C* is considered as a right arch and the portions *AC′C* and *BB′D* would exert only local effects and would be

designed in such a manner as to transfer loads coming upon them laterally across the lines $C'C$ and BB', respectively, into the right arch portion of the structure.

Consider now Fig. 2 (b) in which a line drawn perpendicular to the abutment AB at B passes through the extremity C of abutment CD. In this case the portion of the structure which was a right arch in Fig. 2 (a) has now disappeared and the stability of the structure will depend upon its

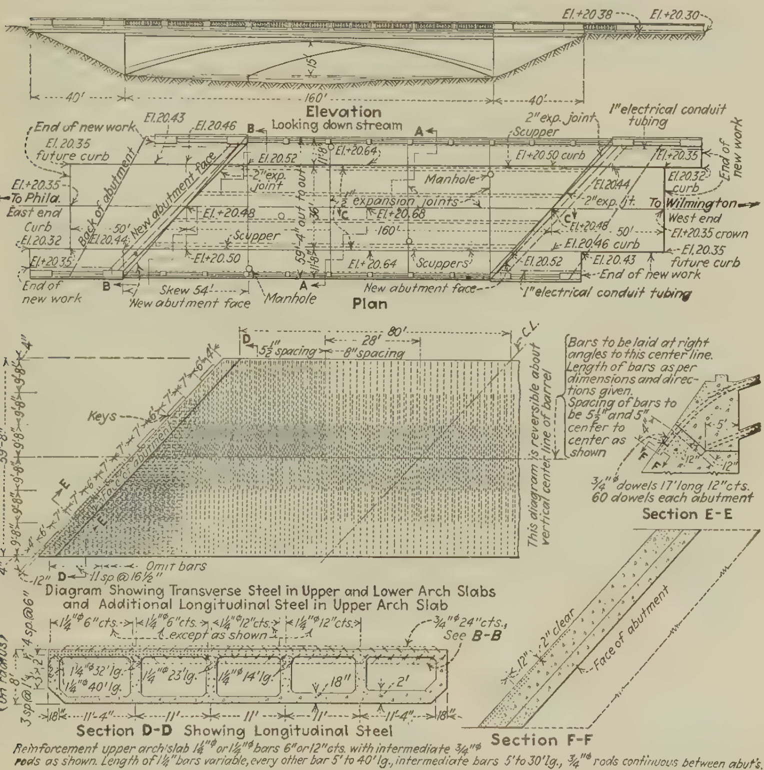


FIG. 3.—ELEVATION, PLAN AND DETAILS.

ability to resist the torsional effects of such an extreme skew. In Fig. 2 (c) the skew condition is even further exaggerated because the perpendicular to the abutment AB drawn through B does not strike the abutment CD at all but passes beyond the end C.

It should be plain from the foregoing that the skew effect is not due solely to the angle of skew but rather is due to a combination of angle of skew with breadth of roadway and length of span. Thus, although the

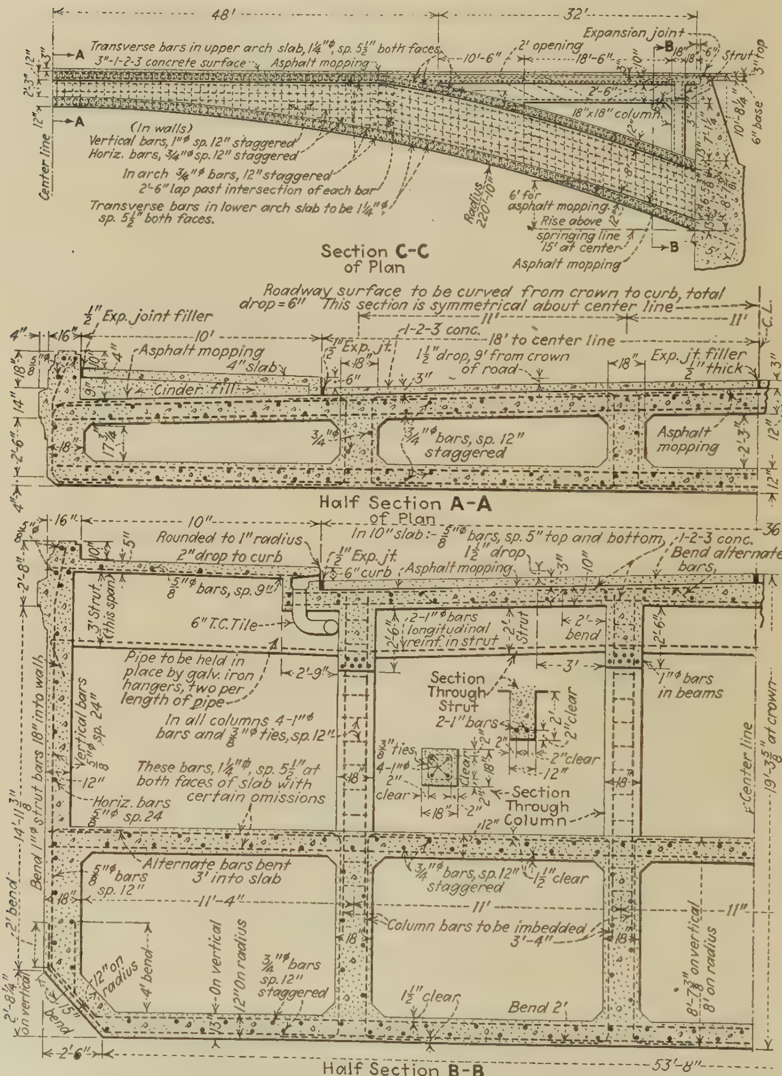


FIG. 4.—TRANSVERSE AND LONGITUDINAL SECTIONS.

angle of skew in the given structure may be large the resulting condition may be one of "partial skew" as in Fig. 2 (a) rather than "total skew" as in Fig. 2 (b) and (c), depending upon the additional factors of length and breadth of span.

In cases of "total skew" the resultant pressure line may cut the abutment outside of its middle third thus producing at the obtuse angles *B* and *C* horizontal and vertical reactions more than double the average

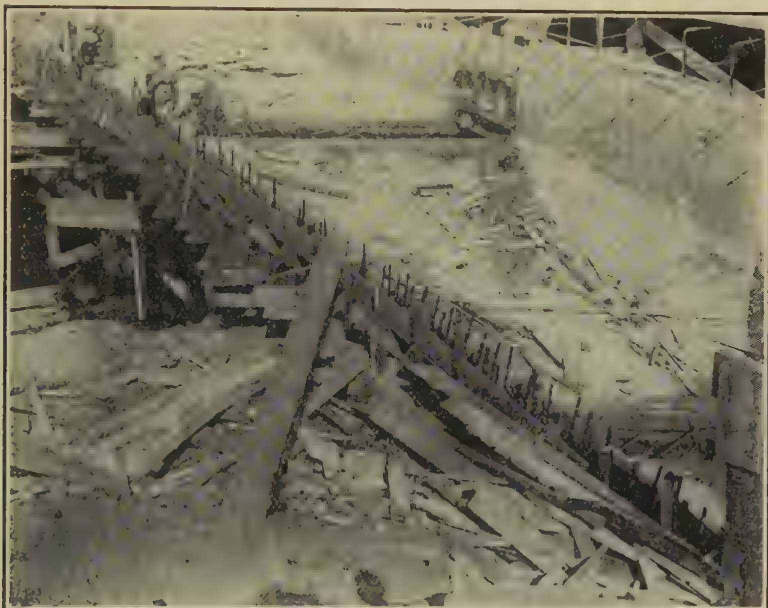


FIG. 5.—JOINTS IN SOFFIT SLAB AND RIBS.

reaction and at the acute angles *A* and *D* even produce tension instead of compression. It is plain also that under such conditions it is not possible to design a skew arch by analysis as a right arch of one lamina parallel to the spandrel even though the loads on the structure be only vertical and arranged symmetrically on the center line of the roadway.

In designing a skew arch of "total skew" as shown in Fig. 2 (b) the reactions at *B* and *C* should be provided in intensities equal at least to twice the average intensity over the whole abutment; and provision should be made at the acute angles *A* and *D* for reversal of horizontal and vertical reaction from compression to tension.

In designing structures of extreme "total skew" such as is shown in Fig. 2 (c) provision must be made for tension at the acute angles *A* and *D* which will surely be present both in uplift and in horizontal reaction;

and the obtuse angles B and C will correspondingly be loaded with thrust and vertical reactions more than twice the average over the abutment. In both of these cases the abutments should be investigated for possible shear across the acute angles of the abutments back of the points B and C. Failures of abutments at these points are known to have taken place in some structures due to the high pressures transmitted from the obtuse angles of the span.

The writer is of the opinion that the suggested provisions for occurrence of tensile stresses in the region of the acute angles should be adhered to, as a precaution against stresses induced by conditions sometimes not possible to foresee..

Great care should be exercised in the case of a thin, wide crown section, since it may cure transversely. This action will affect the analysis for thrusts and moments, if the latter are determined on the assumption that any cross-section retains its form during the loading process.

DESIGN.

Location Conditions.—Ninth St. is 60 ft. in width with a 36-ft. roadway. The gradient at the bridge is so small as to be practically level, although actually there is a slight drainage toward Chester River from both directions. Traffic on this street is very heavy because it not only serves as one of the main arteries of the city of Chester but in addition is a part of the trunk highway system between Philadelphia and Wilmington, Baltimore and Washington. Chester River is crossed by Ninth St. at an angle of about 45 deg. The crossing is about a half mile from the Delaware River into which Chester River empties. Tidal variations at the crossing average about 3 ft. The great breadth of the Delaware rapidly absorbs any sudden increase in runoff from the head waters of Chester River and hence such high water as would be experienced at the crossing would be caused practically entirely by water backing up from the rise in the Delaware itself. The fluctuation in flood water stage in the Delaware at Chester is very small. The elevation of the street gradient is 19 ft., approximately, above the mean water level of the stream.

For more than thirty years a through Pratt truss bridge has carried the street over the stream. This old bridge had a span of 162 ft. with a skew of 55 deg. The county commissioners desired a deck structure in the new construction—that is, they desired no superstructure above the roadway except the hand rails. The general requirements therefore called for some form of deck structure 60 ft. in width, 160 ft. in span with a skew of approximately 45 deg., the whole structure to be built within the 19 ft. between roadway and water surface in such a manner as to allow adequately for proper waterway. A single span structure was essential because small craft navigate to points above the bridge and such navigation is protected through control of the stream by the War Department. Fifteen-foot clearance has heretofore been maintained above mean water level at the center of the channel. Obviously under such conditions some form of arch structure was the only solution.

Determination of Type of Arch.—A study of the conditions as above related disclosed the fact that the minimum amount of skew possible to accommodate the stream was about 40 deg. and that even under such minimum conditions together with a width of 60 ft. and a span of 160 ft., no part of one abutment would lie directly oposite any part of the other abutment but on the contrary normals drawn from the extremities of one abutment face would be no nearer than about 25 ft. to the other abutment. Thus the problem would be one of "total skew" [see Fig. 2 (c)].

FIG. 6.—NOTE KEYWAYS IN ABUTMENT SEAT, ALSO REINFORCEMENT FOR TENSION AT ACUTE ANGLE OF SPAN.

The total depth of the structure at the crown, maintaining the clearance beneath the bridge of 15 ft. and at the same time preserving the easy gradient of the street, was 4 ft. 3 in. Whatever the form of arch, therefore, the flatness of the curvature and the small distance between roadway and soffit at the crown would require at least the middle third of the span to be so constructed as to have the roadway supports and the arch cast monolithic.

The open-spandrel construction in which the floor system is not cast monolithic with the ribs at the crown can be so arranged as to remove entirely the various effects found in the skewed barrel arch. Under such circumstances the ribs would be designed and constructed separately as

right arches and the spandrel and floor framing would consist of bents resting upon these right arches. In the case at hand, however, this scheme could not be utilized. Obviously, an open-spandrel structure built within the narrow limits permitted in the present case would result in a span subjected to severe torsional effects with lateral and torsional structural rigidity absent. For this reason the writer eliminated the open-spandrel type from further consideration.

The cantilever structure was next considered. The great length of span with small rise would result in large distortions due to temperature change. Strains due to this cause and to dead- and live-load would develop statically indeterminate conditions similar to a hinged skew arch due to the skewed abutments. Furthermore, a cross-section made up of longitudinal ribs with a deck slab would not develop torsional stiffness. It seems questionable that the necessary rocker joint which would have to be constructed on the piers within the tidal range could be so constructed as to be reliably efficient throughout a long structural life. Further consideration of the cantilever form was therefore abandoned.

Consideration was next given to the solid barrel type of skew arch. Assuming first of all that such a warped slab could be properly designed to resist torsional stresses, various methods were reviewed for supporting the deck. Transverse walls placed parallel to the abutment face were considered undesirable in that they did not contribute to the strength of the arch on the one hand and on the other imposed upon the warped barrel dead- and live-loads along fixed lines at intervals along the span. Column supports would tend to concentrate these dead and live deck loads at fixed points on the barrel without adding to the structural strength of the barrel itself. Bearing in mind also that the total thickness of the bridge at the crown was limited to 4 ft. 3 in. out of which both deck slab and barrel thicknesses would have to be taken, there would result a region of considerable stiffness in the central part of the span which would not properly or along direct paths be carried to the abutments. This would especially be true of the torsional moments.

Longitudinal ribs seemed more favorable to meet the various conditions in that they would add to the structural stiffness and would not tend to load the barrel at a series of panel points. However, in the region of the haunches these walls would be quite high and the effect of their stiffness upon arch action together with the deck slab which would be cast monolithic with them would require a thorough anchoring of both walls and deck slab at the abutments to resist bending about the horizontal or oblique axis. Plans were prepared on this design but upon further consideration this form was rejected in favor of a more flexible construction in the region of the haunches obtained by making the deck structure on this area entirely separate from the arch and by forming the arch for its full length of two warped slabs separated by longitudinal ribs (Fig. 4).

The tubular construction finally adopted appeared to permit the most economical use of materials in the middle portion of the span, in giving

proper strength to the deck slab and at the same time employing this deck slab together with the soffit slab in developing a maximum of torsional stiffness for the amount of material employed. Furthermore, continuation of this central portion of the deck slab on an incline to the abutment to serve as the upper face of the tubular arch made possible carrying the stresses set up by torsional moments over the most direct paths to the abutment. A certain continuity of torsional resistance was thus accomplished throughout the length of the span. It should be borne in mind that

FIG. 7.—TORSION REINFORCEMENT IN EXTRADOS SLAB.

when a long rectangular section is submitted to torsional strain, the stress is a maximum at the middle of the long side of the rectangle.

Construction Conditions.—In order to obtain the greatest possible rise to the arch, the springing line was put at about the elevation of mean water level. To support such an arch during construction would require piling driven in the mud bottom of the stream to refusal on the rock below. The depth to rock at the face of the west abutment was about 3 ft. and at the east, about 17 ft. The mud above, however, could not be depended upon to resist lateral forces. There was likelihood, also, of winter high water sufficient to submerge the arch haunches, and perhaps weaken the falsework structure. Because of these conditions, it was essential to arrange the construction and design to accommodate a falsework structure in which lateral rigidity could not be depended upon to any great extent.

It was realized in the design that the sequence of construction would be: First, to cast the soffit slab of the arch; second, to cast the ribs; and third, to cast the extrados slab. In this sequence, it was also appreciated that if the soffit slab were cast complete, the settlement of the falsework when the ribs were cast would introduce stresses in the soffit slab and thus probably introduce into the falsework oblique thrusts. Furthermore, as the extrados slab was later added there would be further stresses introduced into both soffit slab and falsework. These various conditions would impose stress distribution in the arch quite different from that indicated in an analysis in which the stress over the entire arch cross-section would be in accordance with the distribution of moments and thrusts.

In the construction, lateral stiffness was not provided for the falsework, so the soffit and ribs were cast in five large voussoirs, with narrow gaps between; and these large sections were permitted to cure for about two weeks before the gaps were concreted. Thus the arch effect was not allowed to develop stresses in either the falsework or the soffit slab and ribs, until the latter were in place and had gained considerable strength. There was also provided, by this method, greater structural strength before the actual concreting of the extrados slab (Fig. 5).

Design.—The various theories advanced for the analysis of a skew arch have until recently failed to provide for transverse moment and shear and the consequent non-uniform distribution of stress along the abutment face. The result has been that there have been several failures of such structures and therefore a general distrust in the structural integrity of a skew arch. A more advanced treatment is given by Prof. Rathbun.* While not willing to subscribe to Prof. Rathbun's conclusion (that for skew arches loaded vertically and symmetrically the analysis can be made as for a right arch of span equal to the skew span), the writer finds much of value in the paper.

The effect of torsion on a cross-section of a skew arch may also be found in a right arch which is loaded unsymmetrically across its width. Such is the case of a double-track railroad bridge with only one track loaded. Such effect is neglected in ordinary design of right arches, although for some cases this should be studied. No published theory is at hand to make such a study.

Torsion on a cross-section of an arch is of interest because it involves an action on which we have little or no test data. It is believed there is more justification for dependence upon the tubular cross-section than upon the thin, slender cross-section of a solid barrel arch, in the light of our present knowledge of such sections under torsional strain. The latter form of section is not only a poor form for torsional stiffness, but is also subject to bending tranversely across the bridge for all distributed loadings.

The bridge was analyzed as a skew arch, fixed at the abutments. The span was cut at the crown parallel to the abutment and the half-span analyzed as a curved space surface. Linear and angular deflections at the

* Am. Soc. C. E. *Transactions*, 1924.

crown were determined for each half-span and equated. Investigations were made for loadings symmetrical about the roadway center line, hence only four equations of condition resulted, involving the four unknowns—horizontal thrust, transverse shear, and bending about the two horizontal axes.

Disposition of the torsional stresses was done by means of the formula for a hollow rectangle reduced from the analysis by Saint-Venant. The writer would not feel justified in applying a similar analysis to a long, slender cross-section of a solid barrel arch.

The soffit slab was increased in thickness in the region of the abutment at the obtuse angle of the span, to provide for the large compression at that place (Fig. 4).

Keyways were constructed in the abutment seat to provide against the tendency to slip along the abutment because of the transverse shear (Fig. 6). Steel bars were embedded in the abutments at the acute angle of the span, to provide for the tendency of the span to lift up, and pull away, from the abutment seat in that region (Fig. 6). Computations were made to determine the stresses in the cross-section of the ribs because of torsion. Investigation was also made into the stresses in the various panels of arch slab between ribs, due to the same torsional moment.

The analysis showed the need for a large amount of tranverse steel to provide against the diagonal tension arising from the torsion on the span's cross-section. The amount of this steel was greatest in the central bay of the arch slab; and each succeeding bay required less. The variation of shearing stress along the cross-section of the arch was taken as the ordinates to a parabola whose middle ordinate was at the center of the central bay and equal to the stress at that point, and whose curve cut the outward corners of the cross-section with zero ordinates. Torsional shearing stress in the ribs was computed to be proportionate to the distance of the rib from the center of the cross-section, but the stress in the outer rib was determined as for a hollow rectangle, from the application of Saint-Venant's analysis. The steel in the arch slabs for resisting torsion was placed at right angles to the roadway. For combined torsional shear and local bending, the working unit stress was limited to 16,000 lb. per sq. in. (Fig. 7).

CONSTRUCTION.

Abutments.—The construction of the west abutment was attended with no unusual features. Rock of good quality was found close to the springing level. Irregularities were easily cut into the rock to resist sliding and rotation in the various directions indicated in the design. Borings failed to disclose a condition in the rock surface at the site of the east abutment. Rock was anticipated and found about 50 ft. below the street level or about 35 ft. below low water. The rock surface fell away in the manner shown in Fig. 8 so that a thrust bearing was not available.

A careful examination was made of the rock surface to determine its formation and soundness. It was found to be a fair quality of granite

bordering on gneiss. It was decided to anchor dowels into the ledge and cap it with the concrete of the abutment; 9-ft. 1¼-in. square dowels were used for this purpose. Holes 3 in. in diameter were drilled spaced from 18 in. to 3 ft. each way, depending upon the location. The holes were sloped downward toward the front (Fig. 8). Along the upper shelf vertical holes were also drilled for dowels. The bars were grouted into the holes for half their length. This work is shown in Fig. 9 which is a photograph looking down into the excavation. The rock ledge was carefully cleaned and washed before placing the concrete of the abutment.

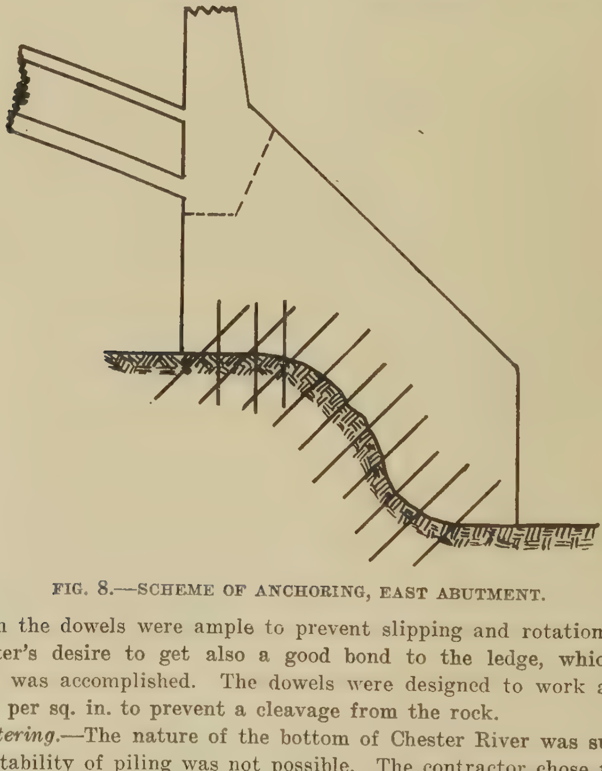


FIG. 8.—SCHEME OF ANCHORING, EAST ABUTMENT.

Although the dowels were ample to prevent slipping and rotation, it was the writer's desire to get also a good bond to the ledge, which, it is believed, was accomplished. The dowels were designed to work at about 5,000 lb. per sq. in. to prevent a cleavage from the rock.

Centering.—The nature of the bottom of Chester River was such that lateral stability of piling was not possible. The contractor chose to adopt the writer's alternate of casting the arch in voussoirs rather than construct a system of lateral struts to brace the piling. The centering, therefore, did not differ from the usual centers for a right arch. An open channel was maintained at the center of the span by heavy I-beams spanning from a bent on one side of the channel to one on the other (Fig. 10). The beams were bridged to prevent overturning. No attempt was made to procure a simultaneous striking of all parts of the centering. Instead, the removal of wedges accompanied the lowering and removal of the sheathing

and was in progress over about three days. The operation commenced at the crown and progressed both ways.

Concrete.—It was desired to obtain an exterior finish in which the aggregate would be revealed. Care was exercised therefore in the selection from commercial supplies submitted by the contractor. A river gravel locally obtained was finally selected. As to appearance it was predominantly buff, with shades of colors interspersed, running out to deep purple, dark red, brown and black. The color mixture was quite uniform. This colored aggregate did not add any extra cost to the structure (Fig.

FIG. 9.—DOWELS BEING GROUTED INTO ROCK LEDGE.
Viewed from back of abutment facing arch, and looking down.

13). Preliminary tests had been made on concrete using this aggregate prior to concreting operations, and from these a proportion of 1: 2.33: 3.84 had been determined as the one assuring 2,000-lb. per sq. in. in 28 days, when placed at the consistency required in the arch members. The concrete in the abutments was in the proportion of 1: 3.5: 5.75. Sound clean stone removed from the old masonry abutment were placed by the derrick into the concrete mass of the abutment to an amount of about 35 per cent of the entire bulk of the abutment. The back slope was formed by laying up stones in layers as the concrete proceeded. The abutment concrete was sufficiently dry not to bulge the stones thus placed out of position.

Concreting.—The procedure in casting the arch was, first, to cast the soffit arch slab; second, the intermediate ribs; third, the outside ribs together with the spandrel walls; fourth, the extrados slab. Then followed the completion of the deck structure over the haunches.

The soffit slab of the arch was cast in five sections, the sections separated by openings 1 ft. wide extending the full width of the barrel on lines parallel to the abutment face (Fig. 5). A tongue-and-groove joint was made between each section of the arch and the concrete subsequently to be placed in the 1-ft. opening. The span of the arch was divided up

FIG. 10.—CENTERING STRUCK. STRUCTURAL DECK IN PLACE. FINISHING WATER-TABLE IN PROGRESS.

into 18-ft. sections adjacent to the abutment, a crown section 70 ft. long, and one intermediate section on either side of the crown section 27 ft. long. The abutment sections were cast first, then the crown section and lastly, the intermediate sections, the concrete being placed on June 10 to 13, inclusive. The next step was the concreting of the four intermediate vertical ribs of the arch. Openings extended vertically upward through the ribs at points directly over the 1-ft. openings in the soffit slab (see Fig. 5). Concreting of the ribs was done between June 21 and June 28. During the process of concreting the soffit slab and ribs the centering was carefully watched by means of telltales for movement or settlement. The amount of movement for settlement actually obtained was negligible.

On June 29 the openings in the soffit slab and intermediate vertical ribs (Fig. 5) were concreted. As has previously been described, it was feared that the dead-load of the bridge during the various stages of concreting would cause settlement in the centering and thus introduce premature skew stresses in the arch which the centering would be unable to resist. The procedure adopted achieved the placing of the dead weight of the soffit slab and intermediate ribs without the development of a skew thrust and then when closure was actually made the additional dead weight in effecting the closure was not sufficient to disturb the equilibrium of the centering. It

FIG. 11.—RIBS AND EXTRADOS SLAB CONSTRUCTION.

will be noted that the soffit slab concrete was two weeks old at the time of closing the joints and that the arch centering had been well tested for rigidity by the concreting operations up to that time.

The next operation of concreting consisted in the casting of the outside spandrel walls from the bottom of the chamfer to the underside of the water-table and for the full length of the spandrel. Since these walls were to have the exposed surface scrubbed to reveal the aggregate, care was exercised in maintaining as nearly as possible a uniform consistency and thorough mixing. Ramming of the concrete in the forms was permitted in the center of the mass rather than next to the face of the forms. The following day the exterior forms were removed and scrubbing was begun

at once on the green concrete, using steel bristle brushes. The concreting of the spandrel walls was done on June 27 and 28.

The next operation was the concreting of the top slab or extrados slab of the arch which was accomplished on August 2 to 5, inclusive. Up to the time of this concreting operation, no great amount of dead weight had been added to the centering subsequent to the closing of the joints in soffit slab and intermediate ribs. The concrete in the soffit slab was now six weeks old and the concrete in the intermediate ribs and the various joints was one month old. Any distortion of the centering due to further

FIG. 12.—COMPLETED ARCH RING, VIEWED FROM EAST ABUTMENT.

loading by additional concreting would be distributed and considerably resisted by the soffit barrel and intermediate ribs, which members had acquired a strength of 2,000 lb. per sq. in. or more.

The concreting of the top slab of the arch was done in five sections, similar to the concreting of the soffit slab. No narrow joints were left, however, in this operation but the succeeding sections were cast against the preceding ones. On the conclusion of this operation the appearance of the structure was that shown in Fig. 12.

The next operation consisted in placing the roadway deck over the haunches, thus completing the structural deck or floor of the bridge. This

was followed by the construction of the water-table (Fig. 10) followed by sidewalks, pavements and balustrade (Fig. 13).

The casting of the spindles of the balustrade is worthy of mention. A set of iron molds was procured at an early stage of the bridge construction. Each day thereafter the molds were filled and on the following day they were opened and the green spindles removed and the molds refilled. The spindles were not reinforced. The molds were filled with concrete mixed quite wet so as to insure the thorough filling of all portions of the mold. Dry coarse aggregate was then rammed into the mold a little at a

FIG. 13.—HAND RAIL DETAIL.

time and any excess water or grout was allowed to overflow the top of the mold. This continued until no additional coarse aggregate could be rammed into it. The top of the mold was then struck off and several thicknesses of newspaper were laid over the top on which was piled previously dried sand. Moisture began to be drawn by the capillarity of the paper, the sand serving as a reservoir for moisture thus drawn. When the sand became quite wet it was scrapped off without disturbing the newspaper and replaced by dry sand. This continued until the drawing of the moisture practically ceased. This is a method the writer learned from John J. Earley. The concrete resulting from such a procedure has a maximum of coarse aggregate and sufficiency of binding material and at the

moment of final set of the concrete has a minimum of water present in the mass. It develops a surprisingly strong concrete, equivalent to that obtained at the high point of the Abrams water-ratio curve. The process insures a rapidly acquired strength at the end of the first day and since there is a maximum of coarse aggregate there is practically no shrinkage, hence no cracking of the spindles at the minimum cross-sections.

EXTENSOMETER MEASUREMENTS.

It was the writer's desire to watch the stresses in the various parts of the arch as construction proceeded in order to detect, if possible, any serious stresses in the arch ring. Fig. 14 shows in plan the scheme of measurements, all of which were taken on the steel reinforcement. One complete set, as shown in Fig. 14, was taken in the top face of the soffit slab; a second set on the lower face of the extrados slab and a third set on the upper face of the extrados slab. By such means it was desired to

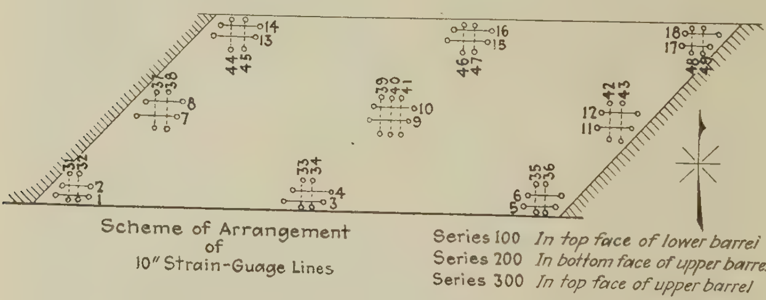


FIG. 14.—SCHEME OF STRAIN-GAUGE LINES.

determine the longitudinal and transverse stresses across the crown cross-section and across a section adjacent to each skewback. Measurements began just before the joints in the soffit slab and ribs were concreted, such measurements forming the datum for all subsequent readings.

The first conclusion from the strain-gage measurements is that the stress increased during construction but at no place to any serious extent. The second conclusion is that the temperature variation plays a very important part in the stresses in such a structure. In fact, temperature variation was at many points on the arch more important in effect than the striking of the centers of the arch ring.

Although many factors influence the strain-gage measurements they nevertheless show the existence of tension in the arch ring at the abutment face in the acute angle of the span. This is further evidenced by a minute hair crack in the spandrel wall between the soffit and extrados slabs close to the abutment face. The details of the strain-gage measurements are shown in Fig. 15.

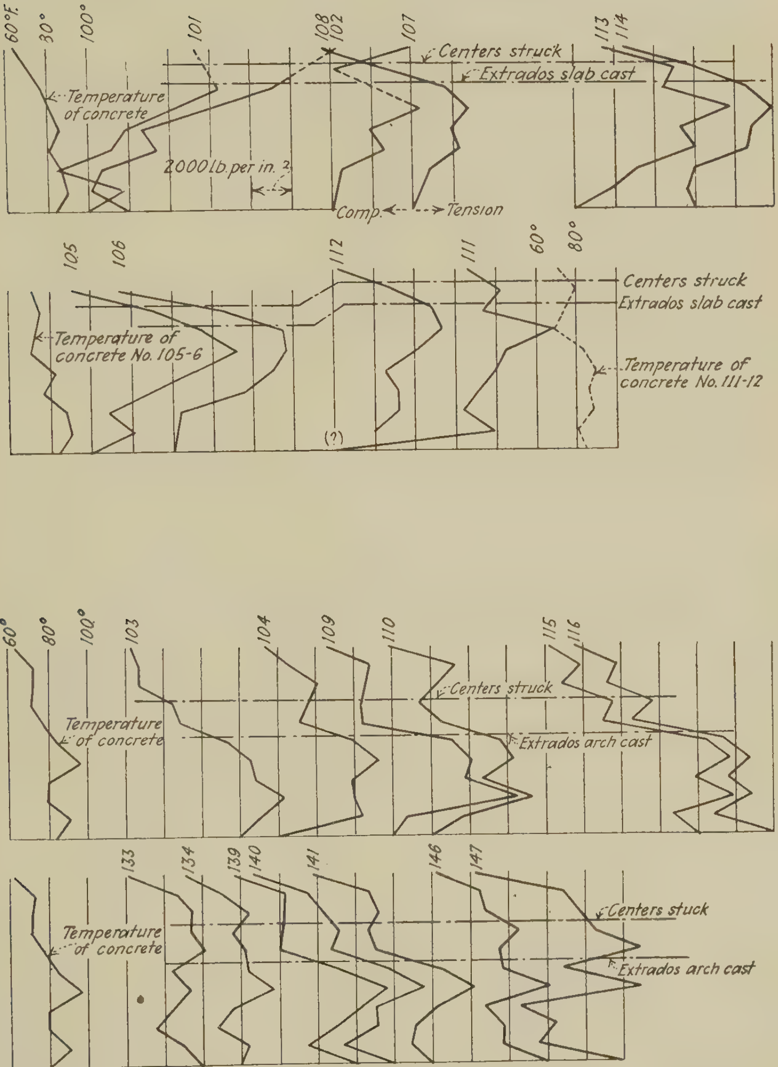


FIG. 15 (A).—STRESSES MEASURED IN REINFORCEMENT IN SOFFIT SLAB DURING AND AFTER CONSTRUCTION.

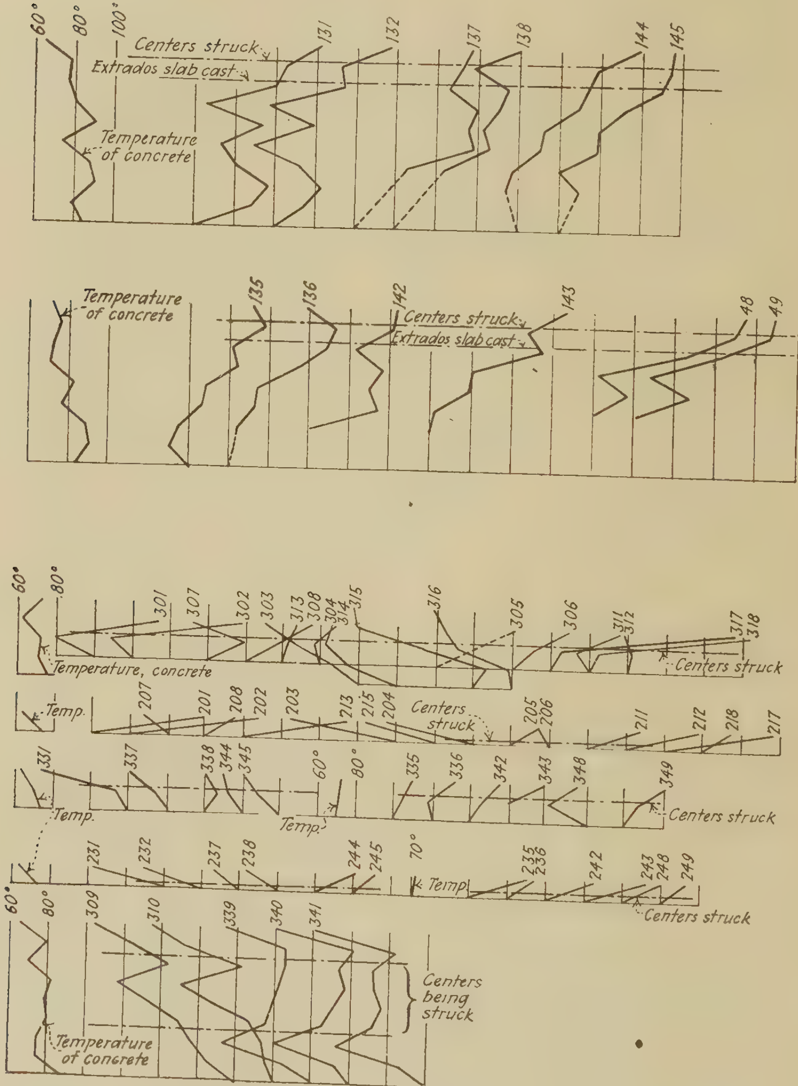


FIG. 15 (B).—STRESSES MEASURED IN REINFORCEMENT IN SOFFIT AND EXTRADOS SLABS DURING AND AFTER CONSTRUCTION.

CONCLUSION.

The useful conclusion to be gained from the experience with the bridge herein described is that the particular skew structure developed sufficient torsional moment and oblique shear to develop tension in the acute angles of the span and that such tension was forecast by the mathematical analysis preceding the design. While the strain-gage measurements offer no accurate quantitative check on the anticipated stresses they nevertheless gave qualitative proof of the forecast given by the mathematical analysis. It seems probable to expect that had the structure been designed as a right arch but having the span of 160 ft. the result would have been a collapse of the arch.

The writer particularly desires to point out the need for further study of wide cross-sections subject to torsion, especially a section of the kind obtained in a solid barrel arch. That such a cross-section will not remain rectangular but will curve considerably under a distributed loading over the arch would indicate a further need for investigation of thin solid barrel skew arches.

ACKNOWLEDGMENT.

The bridge is the property of Delaware County, Pennsylvania, of which the commissioners at the time of construction were Harry M. Birney, Jr., Thomas F. Feeley, and James M. Hamilton. Mundy Paving and Construction Co., of Philadelphia, was the contractor.

The writer is indebted especially to two of his assistants: G. D. Houtman, resident engineer on the bridge, and S. Borge-Poulsen, who joined the staff after the construction had begun, but thereafter made a check of the analysis together with several special studies, and who made all of the strain-gage measurements.

APPENDIX

The design of the bridge described in this paper follows the analysis of a barrel whose cross-section is not likely to bend laterally. In this respect it differs considerably from a thin barrel, which is subject to lateral bending or curling under loads placed between the center-line of the roadway and the spandrel wall.

The following notation will be used (see Fig. 16):

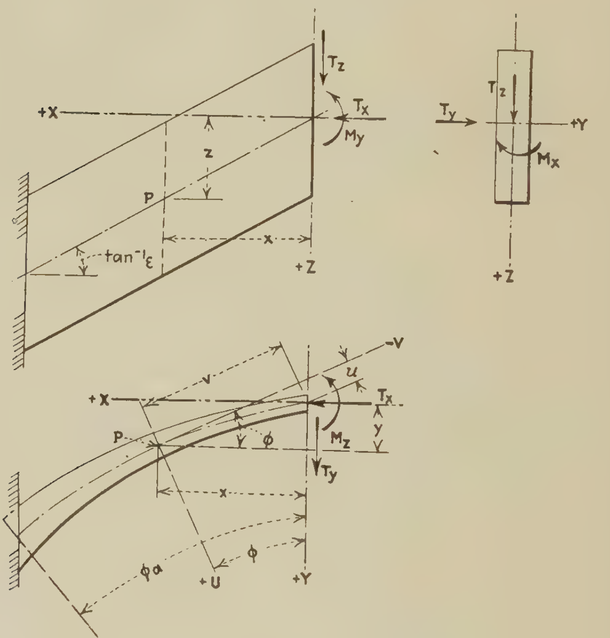


FIG. 16

P = any point on the center-line of the barrel, designated by the co-ordinates x, y, z.

A = area of cross-section at P on radial plane parallel to the Z-axis.

W = a vertical load on the arch barrel, having co-ordinates x_1, y_1, and z_1.

w = uniform live load per square foot of roadway.

λ = ratio of modulus of elasticity in tension and compression to that in shear (E/G).

ϵ = tangent of angle of skew, measured as the angle between the center-line and a normal to the abutment face.

ϕ = angle between vertical through crown and radial line through P, measured parallel to plane X, Y.
I_1 = moment of inertia of area A about horizontal axis (parallel to Z-axis).
I_2 = moment of inertia of area A about radial line through P.
F = torsion factor for area A.
κ = shear factor for area A.
I_x, I_y, I_z = components of crown thrust in the directions X, Y and Z, respectively.
M_x, M_y, M_z = moments at the crown about the axes X, Y and Z, respectively.
T_{ua}, T_{va}, T_{za} = components of abutment thrust in the directions U, V and Z, respectively, for the skewback.
M_{ua}, M_{va}, M_{za} = moments at the abutment skewback about the axes U, V and Z, respectively.
U = axis formed by radial through P.
V = axis normal to U parallel to $X - Y$ plane.
u, v = co-ordinates of crown centroid, referred to the U, V axes.
l = length of span measured parallel to X.
b = Width of barrel measured parallel to Z.
C_1, C_2, etc. = coefficients, as defined later.
L_1, L_2, etc. = load terms, as defined later.

Considering an unsymmetrical arch, unsymmetrically loaded, the equations of condition are as follows:

$$\left.\begin{aligned}
C_1T_x + C_2T_y + C_3T_z + C_4M_x + C_5M_y + C_6M_z &= -L_1 \\
C_2T_x + C_7T_y + C_8T_z + C_9M_x + C_{10}M_y + C_{11}M_z &= -L_2 \\
C_3T_x + C_8T_y + C_{12}T_z + C_{13}M_x + C_{14}M_y &= -L_3 \\
C_4T_x + C_9T_y + C_{13}T_z + C_{16}M_x + C_{17}M_y &= -L_4 \\
C_5T_x + C_{10}T_y + C_{14}T_z + C_{17}M_x + C_{19}M_y &= -L_5 \\
C_6T_x + C_{11}T_y + C_{21}M_z &= -L_6
\end{aligned}\right\} \ldots\ldots \text{Eq. (1)}$$

The left-hand portion of the equations are dependent only upon the form and nature of the arch, while the right-hand terms are dependent upon the loads applied. The coefficients C_1, C_2, etc., and the load terms L_1, L_2, etc., will be defined later.

If the arch is symmetrical, but the load arrangement is unsymmetrical, there will be a cancellation of certain terms, resulting in the following:

$$\left.\begin{aligned}
C_1T_x + C_3T_z + C_4M_x + C_5M_z &= -L_1 \\
C_7T_y + C_{10}M_y &= -L_2 \\
C_3T_x + C_{12}T_z + C_{13}M_x &= -L_3 \\
C_4T_x + C_{13}T_z + C_{16}M_x &= -L_4 \\
C_{10}T_y + C_{19}M_y &= -L_5 \\
C_6T_x + C_{21}M_z &= -L_6
\end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Eq. (2)}$$

A special case of Equation (2) is that of a right arch with loads placed unsymmetrically both longitudinally and laterally. The equations for this special case follow:

$$\left.\begin{aligned} C_1T_x + C_6M_z &= -L_1 \\ C_7T_y &= -L_2 \\ C_{12}T_z + C_{13}M_x &= -L_3 \\ C_{13}T_z + C_{16}M_x &= -L_4 \\ C_{19}M_y &= -L_5 \\ C_{21}M_z &= -L_6 \end{aligned}\right\} \quad \text{Eq. (3)}$$

from which may be found

$$T_x = \frac{-C_{16}L_1 + C_6L_6}{C_1C_{21}} \quad \text{Eq. (4)}$$

$$T_y = -\frac{L_2}{C_7} \quad \text{Eq. (5)}$$

$$T_z = \frac{-L_3 - C_{13}M_x}{C_{12}} \quad \text{Eq. (6)}$$

$$M_x = \frac{-C_{13}L_3 + C_{12}L_4}{C^2_{13} - C_{12}C_{16}} \quad \text{Eq. (7)}$$

$$M_y = -\frac{L_5}{C_{19}} \quad \text{Eq. (8)}$$

$$M_z = -\frac{L_6}{C_{21}} \quad \text{Eq. (9)}$$

In using Eqs. (4) to (9) it should be borne in mind that the curling effect across the arch is not considered in these equations.

Returning now to the skew arch, Eq. (2) may be simplified for the case of the symmetrical arch symmetrically loaded. These equations cover most conditions of skew arch design:

$$\left.\begin{aligned} C_1T_x + C_3T_z + C_4M_x + C_6Mz &= -L_1 \\ C_3T_x + C_{12}T_z + C_{13}M_x &= -L_3 \\ C_4T_x + C_{13}T_z + C_{16}M_x &= -L_4 \\ C_6T_x + C_{21}M_z &= -L_6 \end{aligned}\right\} \quad \text{Eq. (10)}$$

The coefficients for the left-hand terms of Eqs. (1), (2), (3) and (10) are as follows:

$$C_1 = \int\frac{cos^2\phi ds}{A} + \lambda\int\frac{\kappa_2 sin^2\phi ds}{A} + \int\frac{y^2ds}{I_1} + \epsilon^2\int\frac{x^2cos^2\phi ds}{I_2} + \epsilon^2\lambda\int\frac{x^2sin^2\phi ds}{F}$$

$$C_2 = \int\frac{sin\,\phi\,cos\,\phi\,ds}{A} - \lambda\int\frac{\kappa_2\,sin\,\phi\,cos\,\phi\,ds}{A} - \int\frac{xyds}{I_1} + \epsilon^2\int\frac{x^2\,sin\,\phi\,cos\,\phi\,ds}{I_2}$$
$$- \epsilon^2\lambda\int\frac{x^2\,sin\,\phi\,cos\,\phi\,ds}{F}$$

$$C_3 = -\epsilon\int\frac{vx\cos\phi\,ds}{I_2} - \epsilon\lambda\int\frac{ux\sin\phi\,ds}{F}$$

$$C_4 = \epsilon\int\frac{x\sin\phi\cos\phi\,ds}{I_2} - \epsilon\lambda\int\frac{x\sin\phi\cos\phi\,ds}{F}$$

$$C_5 = \epsilon\int\frac{x\cos^2\phi\,ds}{I_2} + \epsilon\lambda\int\frac{x\sin^2\phi\,ds}{F}$$

$$C_6 = \int\frac{yds}{I_1}$$

$$C_7 = \int\frac{\sin^2\phi\,ds}{A} + \lambda\int\frac{\kappa_2\cos^2\phi\,ds}{A} + \int\frac{x^2ds}{I_1} + \epsilon^2\int\frac{x^2\sin^2\phi\,ds}{I_2}$$

$$+\epsilon^2\lambda\int\frac{x^2\cos^2\phi\,ds}{F}$$

$$C_8 = -\epsilon\int\frac{vx\sin\phi\,ds}{I_2} + \epsilon\lambda\int\frac{ux\cos\phi\,ds}{F}$$

$$C_9 = \epsilon\int\frac{x\sin^2\phi\,ds}{I_2} + \epsilon\lambda\int\frac{x\cos^2\phi\,ds}{F}$$

$$C_{10} = \epsilon\int\frac{x\sin\phi\cos\phi\,ds}{I_2} - \epsilon\lambda\int\frac{x\sin\phi\cos\phi\,ds}{F}$$

$$C_{11} = -\int\frac{xds}{I_1}$$

$$C_{12} = \lambda\int\frac{\kappa_1 ds}{A} + \int\frac{v^2ds}{I_2} + \lambda\int\frac{u^2ds}{F}$$

$$C_{13} = -\int\frac{v\sin\phi\,ds}{I_2} + \lambda\int\frac{u\cos\phi\,ds}{F}$$

$$C_{14} = -\int\frac{v\cos\phi\,ds}{I_2} - \lambda\int\frac{u\sin\phi\,ds}{F}$$

$$C_{15} = 0$$

$$C_{16} = \int\frac{\sin^2\phi\,ds}{I_2} + \lambda\int\frac{\cos^2\phi\,ds}{F}$$

$$C_{17} = \int\frac{\sin\phi\cos\phi\,ds}{I_2} - \lambda\int\frac{\sin\phi\cos\phi\,ds}{F}$$

$$C_{18} = 0$$

$$C_{19} = \int\frac{\cos^2\phi\,ds}{I_2} + \lambda\int\frac{\sin^2\phi\,ds}{F}$$

$$C_{20} = 0$$

$$C_{21} = \int\frac{ds}{I_1}$$

The first term in equations for C_1, C_2 and C_7 applies to rib shortening; the second term applies to shear detrusion in a radial direction; and the first term in the equation for C_{12} provides for detrusion in the Z direction. The detrusion terms may be omitted where the unit stress is low, but the rib-shortening effect should be included for flat arches or for cases in which unit stresses are high.

The detrusion terms include the factors κ_1 and κ_2. Since shearing stress is not distributed uniformly over a cross-section, detrusion computations based on average stress must be multiplied by a corrective factor to account for the variation in stress. For a rectangle, $\kappa = 6/5$; and for a hollow rectangular section, κ is approximately equal to the area of the cross-section divided by the area of the two sides parallel to the direction of shear.

The torsion factor F must be used in place of the polar moment of inertia when the cross-section is not circular. In a rectangular section the shearing stress varies along the boundary lines as ordinates to a parabola; hence for such a shape the maximum stress is at the center of the longer side. According to St. Venant,[1] for a solid rectangle with sides a and b,

$$F = \frac{3b^3d^3}{10(b^2 + d^2)} \quad \ldots\ldots\ldots\ldots (11)$$

From this an approximate formula for a hollow rectangle can be developed:

$$F = \tfrac{3}{10}\left[\frac{b^3d^3}{b^2 + d^2} - \frac{b^3_1d^3_1}{b^2_1 + d^2_1}\right] \quad \ldots\ldots\ldots\ldots (12)$$

In the present design, the detrusion terms were included. Computations were made to determine the effect of rib-shortening upon the results, also the effect of omitting all terms involving I_2 and lateral detrusion.

The six loading terms may be readily set up for any desired load arrangement. For a concentrated load W at the point x_1, z_1:

$$\left.\begin{aligned}
L_1 &= C_2W - C_4Wz_1 + C_6Wx_1 \\
L_2 &= C_7W - C_9Wz_1 + C_{11}Wx_1 \\
L_3 &= C_8W - C_{13}Wz_1 \\
L_4 &= C_9W - C_{16}Wz_1 \\
L_5 &= C_{10}W - C_{17}Wz_1 \\
L_6 &= C_{11}W + C_{21}Wx_1
\end{aligned}\right\} \quad \ldots\ldots\ldots\ldots \text{Eq. (13)}$$

In evaluating the coefficients in the foregoing equations the limits of integration are the abutment and the load. The equations (1), (2), (3) or (10) are combined with Equation (13) for the loaded half of the arch, the former having been applied to the entire span.

The loading equations for a distributed load, such for instance as the dead load, are as follows:

[1] Todhunter and Pearson, "History of the Theory of Elasticity," Vol. II, pt. I,

$$\left.\begin{aligned}
L_1 &= \int \frac{D \sin\ \phi \cos\phi\ ds}{A} - \lambda \int \frac{D_{K_2} \sin\phi \cos\phi\ ds}{A} - \int \frac{M_{DX}\ yds}{I_1} \\
&\qquad + \epsilon \int \frac{M_{DZ}\ x \sin\phi \cos\phi\ ds}{I_2} - \epsilon\lambda \int \frac{M_{DZ}\ x \sin\phi \cos\phi\ ds}{F} \\
L_2 &= \int \frac{D \sin^2\phi\ ds}{A} + \lambda \int \frac{D_{K_2} \cos^2\phi\ ds}{A} + \int \frac{M_{DX}\ xds}{I_1} \\
&\qquad + \epsilon \int \frac{M_{DZ}\ x \sin^2 ds}{I_2} + \epsilon\lambda \int \frac{M_{DZ}\ x \cos^2 ds}{F} \\
L_3 &= -\int \frac{M_{DZ}\ v \sin\phi\ ds}{I_2} + \lambda \int \frac{M_{DZ}\ u \cos\phi\ ds}{F} \\
L_4 &= \int \frac{M_{DZ} \sin^2\ \phi\ ds}{I_2} + \lambda \int \frac{M_{DZ} \cos^2\phi\ ds}{F} \\
L_5 &= \int \frac{M_{DZ}\ \sin\phi \cos\phi\ ds}{I_2} - \lambda \int \frac{M_{DZ} \sin\phi \cos\phi\ ds}{F} \\
L_6 &= -\int \frac{M_{DX}\ ds}{I_1}
\end{aligned}\right\} \text{Eq. (14)}$$

in which D is the vertical load between any point P and the crown section, M_{DX} is the statical moment of D about a line through P parallel to the Z-axis, and M_{DZ} is the statical moment of D about a line through P parallel to the X-axis.

If a vertical uniform load of w lb. per sq. ft. is placed over the half span, the loading terms in Eq. (14) may be used by substituting

$$bwx = D \qquad \frac{bwx^2}{2} = M_{DX} \qquad \frac{bwz^2}{2} = \frac{bwx^2}{2}\epsilon^2 = M_{DZ} \text{ .....Eq. (15)}$$

Temperature loading terms for the half span are as follows:

$$L_{1t} = ltcE \qquad L_{2t} = L_{4t} = L_{5t} = L_{6t} = 0 \text{ ....Eq. (16)}$$
$$L_{3t} = eltcE$$

in which c = coefficient of thermal expansion and E is multiplied by 144 if l is in feet. These equations do not provide for the secondary effects of volumetric change.

Analysis was made of Ninth St. bridge for stresses under dead load and a live-load of 150 lb. per sq. ft. of deck. No allowance was made for the unequal stresses due to the sequential operations of construction, because these effects could not be evaluated. The results, including the effects of thrust and shear, are as follows:

$$\left.\begin{aligned}
T_x &= 3{,}146{,}000 \text{ lb. (Thrust)} \\
T_z &= 1{,}054{,}000 \text{ lb. (Oblique shear)} \\
M_x &= -2{,}770{,}000 \text{ ft.-lb. (Torsion)} \\
M_z &= 9{,}250{,}000 \text{ ft.-lb. (Bending about } Z\text{-axis)}
\end{aligned}\right\} \text{.............(17)}$$

all of which apply to the crown section. The resultant thrust has an eccentricity of 2.94 ft. upward.

The stresses on the U-Z plane at the abutment are as follows:

$$\left.\begin{array}{llll} T_{va} & = & 4{,}270{,}000 \text{ lb.} & = \text{Thrust.} \\ M_{za} & = & -36{,}850{,}000 \text{ ft.-lb.} & = \text{Moment about } Z\text{-axis.} \\ M_{ua} & = & 133{,}300{,}000 \text{ ft.-lb.} & = \text{Moment about } U\text{-axis.} \\ M_{va} & = & 12{,}900{,}000 \text{ ft.-lb.} & = \text{Torsion about } V\text{-axis.} \end{array}\right\} \quad \ldots\ldots\ldots(18)$$

The thrust line cuts this plane at a point 9.2 ft. below, and 31.3 ft. to the right of, its center (toward the obtuse angle of the span).

It is not uncommon, in right-arch design, to omit the rib-shortening effect from the computations. The effect of so doing on skew arch analysis, however, is quite different than on ordinary arches, as the following results for the crown section show:

$$\left.\begin{array}{llll} T_x & = & 3{,}439{,}000 \text{ lb.} & \text{(Thrust)} \\ T_z & = & 1{,}284{,}000 \text{ lb.} & \text{(Oblique shear)} \\ M_x & = & -2{,}165{,}000 \text{ ft.-lb.} & \text{(Torsion)} \\ M_z & = & 8{,}730{,}000 \text{ ft.-lb.} & \text{(Bending about } Z\text{-axis)} \\ e & = & 2.54 \text{ ft. above.} & \end{array}\right\} \quad \ldots\ldots\ldots\text{Eq. (19)}$$

These results show an increase in forces, and a decrease in moments over Eq. (17). The effect of the rib-shortening is, therefore, the reverse of these differences. It is important, in considering time-flow of the concrete under load to note this effect together with the following, for the abutment section:

$$\left.\begin{array}{llll} T_{va} & = & 4{,}525{,}000 \text{ lb.} & = \text{Thrust} \\ M_{za} & = & -33{,}700{,}000 \text{ ft.-lb.} & = \text{Moment about } Z\text{-axis} \\ M_{ua} & = & 136{,}000{,}000 \text{ ft.-lb.} & = \text{Moment about } U\text{-axis} \\ M_{va} & = & 10{,}330{,}000 \text{ ft.-lb.} & = \text{Torsion about } V\text{-axis} \end{array}\right\} \quad \ldots\text{Eq. (20)}$$

The thrust line cuts this plane at a point 7.44 ft. below, and 30.1 ft. to the right of, the centroid.

Transverse shear deformation is of importance in the case of a long span with large angle of skew. The effect on the stresses in the present analysis may be seen by comparing the following results at the crown obtained by omitting the transverse shear term, with Eq. (17) which includes such term:

$$\left.\begin{array}{llll} T_x & = & +4{,}305{,}000 \text{ lb.} & \text{(Thrust)} \\ T_z & = & +3{,}228{,}000 \text{ lb.} & \text{(Oblique shear)} \\ M_x & = & -3{,}516{,}000 \text{ ft.-lb.} & \text{(Torsion)} \\ M_z & = & 7{,}090{,}000 \text{ ft.-lb.} & \text{(Bending)} \\ e & = & 1.65 \text{ ft. above} & \end{array}\right\} \quad \ldots\ldots\ldots\ldots\ldots\text{Eq. (21)}$$

Since I_2 is large when compared with I_1 the term involving I_2 is usually much smaller than that involving I_1. If the present design, however, the value of I_1 is much larger than for an ordinary arch barrel and the I_2 terms are more important. If these terms alone were dropped their effect can be seen by comparing the following with Eq. (17):

$$\left.\begin{array}{lll} T_x = & 3{,}090{,}000 \text{ lb.} & \text{(Thrust)} \\ T_z = & 264{,}000 \text{ lb.} & \text{(Oblique shear)} \\ M_x = & -868{,}000 \text{ ft.-lb.} & \text{(Torsion)} \\ M_z = & 8{,}350{,}000 \text{ ft.-lb.} & \text{(Bending)} \end{array}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

The large reduction in the torsional moment and transverse shear, indicates the undesirable effect of omitting the I_2 terms.

Temperature stresses were computed for the fully loaded structure. Since the average temperature when concreting the arch would be about 80 deg. F., a range was chosen with a drop of 60 deg. and a rise of 10 deg. from that temperature. The results for the crown section follow:

$$\left.\begin{array}{lll} & \textit{Drop of 60 Degrees} & \textit{Rise of 10 Degrees} \\ T_x = & -\ 3{,}298{,}000 \text{ lb.} & +4{,}193{,}000 \text{ lb.} \\ T_z = & -\ 6{,}886{,}000 \text{ lb.} & +2{,}348{,}000 \text{ lb.} \\ M_x = & -\ 9{,}150{,}000 \text{ ft.-lb.} & -1{,}850{,}000 \text{ ft.-lb.} \\ M_z = & +20{,}820{,}000 \text{ ft.-lb.} & +7{,}420{,}000 \text{ ft.-lb.} \end{array}\right\} \quad \ldots\ldots\ldots (23)$$

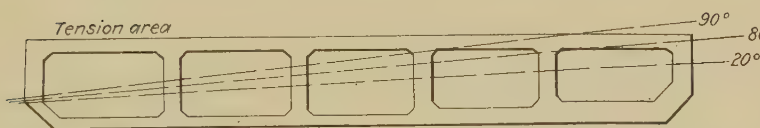


FIG. 17.—EFFECT OF TEMPERATURE ON POSITION OF NEUTRAL AXIS AT ABUTMENT SECTION.

These results should be compared with Eq. (17), whence it will be noted that the stress variation due to winter temperatures is probably the most severe condition to be imposed on the structure. It is true that there is a lag in the rate at which such a structure will achieve atmospheric temperature and therefore a duration of perhaps a week of near-zero weather will be necessary to bring the average temperature of the arch down to 20 deg. F. This is possible, but the likelihood of continuance is rare. Nevertheless, such a condition was provided for in the design.

The abutment section was similarly analyzed with the following results:

$$\left.\begin{array}{lll} & \textit{Drop of 60 Degrees} & \textit{Rise of 10 Degrees} \\ T_{va} = & -\ \ 1{,}575{,}000 \text{ lb.} & +\ \ 5{,}215{,}000 \text{ lb.} \\ M_{za} = & -105{,}800{,}000 \text{ ft.-lb.} & -\ 25{,}600{,}000 \text{ ft.-lb.} \\ M_{ua} = & +243{,}600{,}000 \text{ ft.-lb} & +116{,}200{,}000 \text{ ft.-lb.} \\ M_{va} = & +\ 38{,}300{,}000 \text{ ft.-lb.} & +\ \ 8{,}900{,}000 \text{ ft.-lb.} \end{array}\right\} \quad \ldots \text{Eq. (24)}$$

Comparing Eqs. (17), (18), (23) and (24) it is seen that the maximum torsion at the crown is 9,150,000 ft.-lb., and at the abutment, 38,300,000 ft.-lb., both occurring when the arch temperature drops to 20 deg. F. The average condition produces a torsion at the crown of about 3,000,000 ft.-lb., and at

the abutment, about 15,000,000 ft.-lb. The need of the large amount of transverse steel is thus apparent.

The position of the neutral axis at the abutment section for the three cases given in Eqs. (18) and (24) are shown in Fig. 17, each position marked by the average temperature producing it. The torsional moment and oblique shear on the section are to be added in analyzing the stress condition. It is possible to obtain from Fig. 15 a fair idea of the measured stress distribution on this same section. It would appear from these measurements that the upper right-hand corner lies in the vicinity of the neutral axis. Measurements at the lower left (gage lines 101–2) are less definite because of accumulated construction stresses. It is hoped subsequent measurements will throw further light on temperature and time-flow effects.

Discussion.—Chester River Arch Bridge.

Phil J. Markmann.—The successive methods considered and described for the design of a skew arch bridge over the Chester River, Chester, Pa., have finally led the designer to a design in which the roadway is supported by 6 arch ribs, 4 intermediate ribs of I cross-section and 2 end ribs of channel cross-section, running parallel to axis of roadway and having a clear span of 160 ft. between abutments oblique to the roadway at an angle of 45 deg. (so assumed in this review). It may reasonably be assumed that all dead weight of the concrete structure, all the superimposed roadway pavement and all the superimposed live load is conveyed to the abutments along paths lying in the direction of these arch ribs. Mr. Markmann.

There are 4 intermediate full units and 2 end arch ribs, assumed, for expediency in this review, as half units. Each intermediate unit, as designed, carries

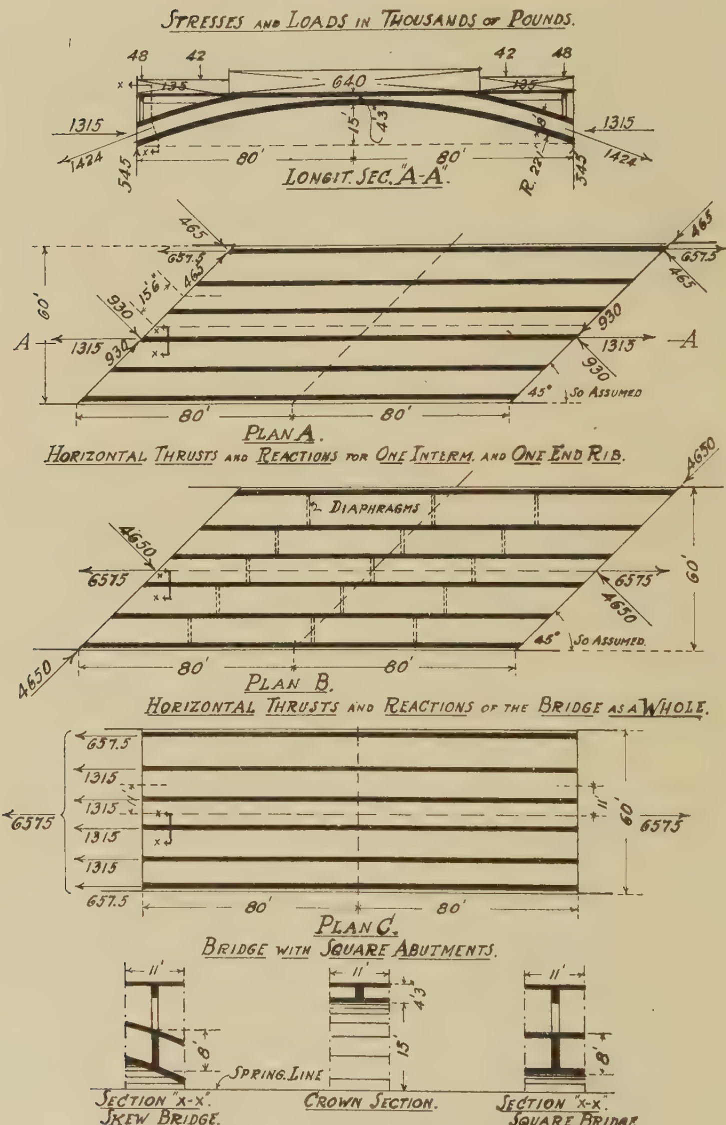
Concrete weight	867,000 lb.	
Pavement weight ...	62,000 lb.	(figured at 35 lb. per sq. ft.)
Live load	161,000 lb.	" " 90 lb. " " "
Total	1,090,000 lb.	

thus bringing 545,000 lb. to the abutment of each rib. The distribution over the 160-ft. span is shown in "Longitudinal Section A-A." The normal total compressive stress on the arch rib next to the abutment is approximately 1,424,000 lb. The horizontal component of this thrust at the abutment, in line with the axis of the rib (and, therefore, the crown thrust also) is approximately 1,315,000 lb. The horizontal thrust at the abutment resolves itself into two components, one normal to the abutment face and the other along the abutment wall. Under the assumption of 45 deg. skew, these components are each 930,000 lb. (see Plan A).

In "Plan B" the 4 full and 2 half units are summed up and give: 6,575,000 lb. as the *total* horizontal thrust against each abutment; 4,650,000 lb. as the *total* resisting component normal to the abutment; and 4,650,000 lb. as the *total* resisting component in line with the abutment.

"Plan C" shows the horizontal forces and the reactions to them for a bridge with right angle abutments of the same clear span. The load supported by each arch rib, the span, the rise of the arch, the cross-sectional area at abutment and crown, the compressive stresses at abutment and crown, and the horizontal reactions to each rib are identical with the respective quantities of the same rib between skew abutments. Each arch rib in this bridge demands a normal reaction by the 11-ft. long abutment of 1,315,000 lb. while the normal reaction by the 15½ ft. long abutment, in the case of the skew span, is only 930,000 lb. There is no *lateral* component of the thrust in the right angle bridge and, therefore, no lengthwise reaction by the abutment wall is called for.

The arch in the right bridge is just as strong as the same arch in the skew bridge, provided the 11 ft. long abutment has about 42 per cent more



CHESTER RIVER TUBULAR ARCH BRIDGE, CHESTER, PA.

strength than the $15\frac{1}{2}$ ft. long abutment of the skew bridge, with respect to shear resistance, overturning moment and width of base. The horizontal reactions to the thrusts, in the case of the right bridge, are shown in "Plan C." A cross section of the arch rib, normal to the arch axis, is shown by Section *x-x* at bottom of diagram. In the case of the right bridge this normal section is right-angled, but of the same area.

The cross sectional area of the arch rib is 31 sq. ft. *at the abutment,* and 1,424,000/31 = 46,000 lb. per sq. ft. or 320 lb. per sq. in.; the cross sectional area of the arch rib is 25 sq. ft. *at the crown,* and 1,315,000/25 = 52,600 lb. per sq. ft. or 366 lb. per sq. in. These are the respective unit compressive stresses, if the "line of pressure" would pass through the mid axis of these cross sections.

The section modulus of the plain concrete cross section, *near the abutment,* is 75 (in ft.). The direct pressure is 46,000 lb. per sq. ft. A bending moment causing an extreme fibre stress of $f = 46{,}000$ lb. per sq. ft. would reduce the concrete stress on the tension side to zero, and would increase the stress on the opposite side to 46,000 plus 46,000 = 92,000 lb. per sq. ft. or 640 lb. per sq. in. The moment corresponding to extreme fibre stress $f = 46{,}000$ lb. per sq. ft. is $M = f \times S = 46{,}000 \times 75 = 3{,}450{,}000$ ft. lb. The eccentricity of the 1,424,000 lb. pressure, is, therefore, for this bending moment,

$$e_1 = \frac{3{,}450{,}000}{1{,}424{,}000} = 2.42 \text{ ft.}$$

The steel in the flange (eleven ¾-in. dia. rods = 4.84 sq. in.) stressed at 20,000 lbs., with a $jd = 6\frac{1}{2}$ ft. furnishes an additional moment of $4.84 \times 20{,}000$ lb. $\times 6\frac{1}{2}$ ft. = 639,000 ft. lb., so that the total resisting moment is 3,450,000 plus 639,000 = 4,089,000 ft. lb., permitting a total eccentricity of

$$e^1_1 = \frac{4{,}089{,}000}{1{,}424{,}000} = 2.87 \text{ ft.}$$

The section modulus of the plain concrete section, *at the crown,* is 28 (in ft.). The direct pressure is 52,600 lb. per sq. ft. A bending moment causing an extreme fibre stress of $f = 52{,}600$ lb. per sq. ft. would reduce the concrete stress on the tension side to zero, and would increase the stress on the opposite side to 52,600 plus 52,600 = 105,200 lb. per sq. ft. or 735 lb. per sq. in. The moment corresponding to extreme fibre stress $f = 52{,}600$ lb. per sq. ft. is $M = f \times S = 52{,}600 \times 28 = 1{,}470{,}000$ ft. lb.

The eccentricity of the 1,315,000 lb. pressure is, therefore, for this bending moment,

$$e_2 = \frac{1{,}470{,}000}{1{,}315{,}000} = 1.12 \text{ ft.}$$

The steel in the flange (eleven ¾-in. dia. rods = 4.84 sq. in.) stressed at 20,000 lb. with a $jd = 3.15$ ft. furnishes an additional moment of $4.84 \times$

20,000 lb. $\times$ 3.15 ft. = 302,000 ft. lb., so that the total resisting moment is 1,470,000 plus 302,000 = 1,772,000 ft. lb., permitting a total eccentricity of

$$e^1_2 = \frac{1,772,000}{1,315,000} = 1.35 \text{ ft.}$$

The permissible eccentricity *near the abutment* ($e^1_1 = 2.87$ ft.) is $\frac{2.87}{8}$ = 36 per cent of the depth of the arch rib, and *at the crown* ($e^1_2 = 1.35$) is $\frac{1.35}{4.25}$ = 32 per cent. of the depth of the arch rib.

These computations show that the arch rib, at the springing line and at the crown, has the required strength to resist the respective pressures having an eccentricity of as much as approximately ⅓ of the respective depths of section.

A positively beneficial feature of the design is apparent from the following reflection: The arch ribs are not individually flanged, the flanges are continuous and monolithic over all 6 ribs. If the 4 intermediate I ribs are carrying equal live loads, the flange width of each would naturally be 11 ft. However, if one of the 4 ribs under the roadway should have a particularly heavy live load, while the adjacent ribs have light live loads, or none at all, the heavily loaded rib would automatically engage a flange width greater than its quota of 11 ft. because the adjacent ribs do not need all of their quota. This will increase the strength of the heavily loaded unit and will approximately equalize the stresses (and with them the deflections) of the 3 arches of this group. Diaphragms (cross ribs between webs of arch ribs), shown in "Plan B," would be desirable to distribute unequal live loads between affected ribs, but in view of the above explained stress equalizing ability of the ribs themselves are not considered so essential.

The writer calls attention to the thrust components acting along the abutments in the skew bridge as shown for the ribs individually in "Plan A," and for the whole bridge in "Plan B." There is little, if any, doubt that each abutment, as a whole, will have a longitudinal strength adequate to resist the whole shear of 4,650,000 lb. acting toward the acute angle *of the span*, in each abutment. (Each abutment, as a whole, has also to resist a shear of 4,650,000 lb. acting normal to its face.) When we look at the acute angle *of the span* we see that the shear by the end rib, along the abutment (465,000 lb.), will in all probability put a tension into the abutment wall back of it, as there does not seem to be a sufficient body of the abutment ahead of it (toward the face of the bridge) to resist this component of the thrust by the end rib. A longitudinal steel reinforcement tying the end portion of the abutment back into the body of this abutment would seem to be indicated.

It has been shown how the eccentricity of the arch pressure (the intercept between the mid axis of the arch rib and the "line of pressure") may cause the steel bars (11 ¾-in. dia. bars in either flange) to be put into tension. The "line of pressure" for a distance of about 25 to 30 ft. from either abutment is "humped" (due to the superimposed load not affecting it in this length), the "line of pressure" in this vicinity will very likely be above the mid axis of the rib causing a greater compression in its upper part and a smaller compression or even possibly a tension in its lower part. This may account for the observed "tension in the arch ring at the abutment face in the acute angle of the span" (see bottom p. 388) rather than "a reversal of horizontal and vertical reaction from compression to tension" (see bottom p. 375).

Conclusion.—If this bridge were built as two separate concentric arch rings, assuming the upper ring to be a true arch just like the lower one, with isolated columns or struts between the two rings to make the two act in unison in supporting the entire load to be carried, then the writer could agree that the problem is that of a skew arch and that the mathematical formulae assembled in the Appendix may or may not give a correct solution. The writer has not taken time to look into the mathematical phase of the skew problem and is not in position to make any comment thereon. However, the flattening of the upper ring in its middle 90 ft. of the 160 ft. span and the introduction of the 6 continuous 18-in. thick reinforced-concrete walls *parallel to the road axis*, between the lower and upper (pseudo) ring, has converted the structure into a system of 6 arch ribs placed alongside of each other, and bridging the waterway with a span of 160 ft. in a direction at an angle with the abutments. The fact that these arch ribs are not merely placed alongside of each other, but are actually monolithic in their flanges does not alter the type of construction, but merely establishes a useful give-and-take system in the co-operation of adjacent arches, as has been stated above. There is not a vestige of skew arch conditions, torsions, etc., in the problem.

The writer was anxious to inform himself how this structure, supposedly computed by the help of the exotic skew formulas in the Appendix, would check up when treated in its proper class. It checks out quite well, there may be a lot of steel in the mass for supposed torsional and uplift stresses which, however, can do no harm. The strength of the arches, as such, is there. The arches, however, depend on the sturdiness of their abutments to which latter the writer has not given any detailed attention, the cross sections of west and east abutments are not recorded definitely enough to invite a check-up.

Mr. Hollister.

S. C. HOLLISTER.—The discussion by Mr. Markmann contains so many inaccurate methods of analysis and concepts of structural behavior that it cannot be passed without serious reply. The closing paragraphs containing his conclusions should be carefully read before proceeding with the earlier part of his discussion.

To begin with, Mr. Markmann "has not taken the time to look into the mathematical phase of the skew problem and is not in position to

make any comment thereon." Without having done this he concludes "there is not a vestige of skew arch conditions, torsions, etc., in the problem." His computations, he says, were made simply because he was "anxious to inform himself how this structure, supposedly computed by the help of the exotic skew formulas in the Appendix, would check up when treated in its proper class."

Now what is meant by the "proper class" of this structure as he conceives it? He believes because (a) the upper ring is flattened for its central 90 ft. of length; and (b) six ribs 18 in. thick are disposed parallel to the axis of the roadway, that the structure is converted into six separate arches (one rib forming each arch) of 160 ft. span placed on oblique abutments. The arches thus formed, he holds, are of two forms of cross-section: the outer ones are of channel form, while the intermediate fours are of I-section. He contends that these ribs act independent of one another, the curved slabs of the arch rings merely providing flanges for the webs of the six arches. He then proceeds to analyze the reactions of one of these ribs in a very sketchy fashion in his "Sec. A-A" and "Plan A." In his "Plan B" he includes a suggestion of cross-diaphragms to assist the slabs in transmitting the loads to the ribs, which, he says, really are the carrying members.

Let us for the moment consider one of the six ribs, which Mr. Markmann contends are really separate arches. If one of these intermediate ribs were loaded as shown in his "Sec. A-A," there would be a shortening of the rib axis due to thrust. Such shortening would be free to take place if the "flanges" of the rib in question were freely cut from adjacent rib "flanges;" but since they are not, there is necessarily produced a shear in these "flanges" parallel to the rib, thus engaging adjacent portions of the structure in carrying the load. Furthermore, since the load deflects the rib vertically, and because it is rigidly attached to the curved slabs above and below, these slabs are thus loaded as are ordinary arch barrels by spandrel walls, and they must then behave as arch barrels and not simply as free flanges of separately performing arch ribs.

Mr. Markmann admits in his conclusions that if the bridge were formed of two concentric arch barrels connected at intervals by columns or struts to cause them to share jointly the vertical arch deflections, then the structure would be of skew type; but on top of this admission he contends that if these columns are replaced by walls, though of even greater efficiency in tying together (to which he would also add cross diaphragms) the two barrels, the structure would no longer be of skew type.

As to the flattening of the central portion of the upper arch slab, there should not be question as to arch performance, since in the general case an arch may be a rectangular or other angular rigid frame without impairing arch action; and in fact, Mr. Markmann himself considers the ribs as arches even though they too are flattened at their centers.

If the structure were not of skew type, why are there tension cracks in the outside ribs at the acute angle only (p. 388) tensile stresses at the

acute angles of the span in both slabs (p. 388) non-uniform distribution of thrust stresses in top and bottom slabs at the abutment face, (Fig. 15, A); stress in the transverse reinforcement (which steel Mr. Markmann says does "no harm") in both slabs at all observed points, (Fig. 15); co-ordinated behavior as revealed by strain-gage measurements for a considerable variation in temperature? None of these could have occurred if Mr. Markmann's hypothesis were observed by the structure. One must not impress one's opinions too rigorously against experimental facts, but rather, must one sit as a student before the performance of the structure itself. Regardless of hypotheses of both designer and critic, the structure may be relied upon infallibly to follow the true laws of mechanics. It is such behavior on this skew structure that the writer has endeavored impartially to report in the course of his paper.

Mr. Beggs.

GEORGE E. BEGGS.—As I understand Mr. Hollister's explanation, he feels that it is rather necessary to have a rib with a hollow cross-section in order to apply with confidence the mathematical theory to the design. I might say that in the analysis of a 30-deg. skew arch in which he used the same proportions of skew arch as was tested by the Bureau of Public Roads in Washington, Professor Rathbun, of the School of Mines at Rapid City, South Dakota, used equations that started out with much the same assumptions as has been shown on the screen. This same arch was tested by experiments at Princeton to determine experimentally what the reaction components would be for any position of load. Then the comparison of the mathematical analysis was made with the experimental analysis, and the agreement was found to be very close, in spite of the fact that we were dealing with an arch that did not have a hollow rib.

The analysis has since been continued for 3 other types of arches of repeatedly narrowing section, so that we finally reached a design which corresponds to the third case shown (one abutment a considerable distance transversely from the other.) In these other arches also the agreement of the mathematical theory was very close to the experiment in spite of the fact that the rib was not hollow. I am wondering if Mr. Hollister thinks or wishes us to feel, as a result of his study and measurements, that it is unsafe to design a skew arch that is not a hollow-rib section?

Mr. Hollister.

S. C. HOLLISTER.—There is no reason for making the section hollow, unless there is likelihood of there being a very thin transverse section at the crown. For instance, if this span had been designed with a 2-ft. thickness at the crown, the total length along the crown section would be 81 ft., and the ratio of 2 ft. to 81 ft. is the slenderness ratio to which I referred.

It may be that in many cases of solid slabs, an analysis assuming an absence of the curling effect would check with the measurements made on the model. On the other hand, there is a question whether such measurements would check if the thickness at the crown were only a small portion of the entire width of the bridge. In the case of the bridge referred to in the paper, the thickness was such that a hollow section was desirable in order to get sufficient depth to provide some semblance of torsional rigidity and to reduce the curling effect. I do not mean to convey the idea that all forms of skew arch should be designed with a hollow section.

The Calculation of Flat Plates by the Elastic Web Method.

By Joseph A. Wise*

1. Introduction.—The history of the development of methods of solution for stresses and deflections of flat plates has been given in many publications[1] and therefore no general account will be given here. Lagrange's equation, which forms the basis for the attempts at exact solutions, has not yet been solved for the general case, and therefore almost all plate problems have been solved by some method of approximation. Nadai[2] and Estenave[3] have given one type of solution by the use of series approximations, but these involve complex formulas containing trigonometric and hyperbolic functions, such that the solutions require a great deal of difficult mathematical manipulation for any but the simplest cases of symmetrical uniform loading. Nielsen[4] has given approximate solutions by the use of difference equations, his solutions being referred to in Westergaard and Slater's[5] paper. Marcus[6] has used the method of difference equations, but has simplified the concepts and made the application of the method more accessible to the engineer without extensive mathematical training by presenting the subject in the form in which this paper will present it.

In order to show the simplicity of the method of the elastic web the subject is introduced by the analagous method of calculating beams by the use of the equilibrium polygon, and particularly the algebraic or difference equation method of calculating stresses and deflections of beams derived from the equilibrium polygon concept. This introduction should serve to make the later procedure and derivations more easily understandable and forms a more natural approach for American engineers than the more devious and mathematical approach of the German literature. The theory of the elastic web and its analogy to the equilibrium polygon is then developed as well as its

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[1] Todhunter and Pearson, History of Elasticity, also other references given below.

[2] Dr. Ing. A. Nadai, Elastische Platten (Julius Springer, 1925).

[3] Estanave, Contribution a l'étude de l'équilibre élastique d'une plaque rectangulaire mince. Theses présentées à la Faculté des Sciences de Paris 1900.

[4] N. S. Nielsen, Bestemmelse af Spoendinger i Plader ved anvendelse af Differensligninger (Copenhagen, G. E. C. 1920).

[5] H. M. Westergaard and W. A. Slater, Moments and stresses in slabs. Proceedings of the American Concrete Institute, 1921.

[6] Dr. Ing. H. Marcus, Die Theorie elastischer Gewebe und ihre Anwedung auf die Berechnung biegsamer Platten. (Julius Springer 1925). (The theory of the elastic web and its use for the calculation of flexible plates.) Dr. Marcus is Director of the Hoch- und Tiefbau-Aktiengesellschaft (Superstructure and Foundation Construction Joint-Stock Corporation) of Breslau.

application to the calculation of stresses and strains in flat plates. The probable accuracy of this method is discussed as well as its applicability to the design of reinforced concrete slabs. This paper will only present the application of the method for the case of square and rectangular slabs freely supported at the four edges, although the method can be generally applied to almost any case of shape, supporting condition, or loading. Dr. Marcus has shown its application for a very great range of conditions, and in very simple, direct fashion. It is hoped that this further development will be presented in future *Proceedings*, so that it will be readily available for American engineers.

This paper makes no claim of originality, except in presentation. Dr. Marcus has very thoroughly covered the subject and very little departure from his procedure seems possible. However, this is believed to be the first statement of the method in the English language and the introduction has been so worded and developed that it is hoped that American engineers will be able to use it readily. This is the sole justification for the paper.

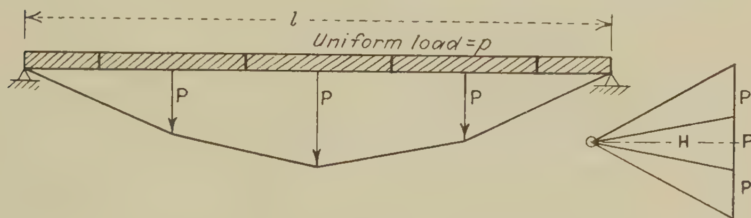

FIG. 1.—EQUILIBRIUM POLYGON, SIMPLE BEAM.

The elastic web is conceived as a network of elastic wires covering the same area as the plate. The loads, if uniformly distributed, are replaced by equivalent concentrations. End conditions representing different conditions of continuity or freedom from restraint at the supports, are determined from the theory of the action of elastic webs (following the theory of thin membranes). The deflections of the web under various loading conditions give moments, stresses and deflections of the plate, much as the equilibrium polygon can be made to yield analagous quantities for beams. The deflections of the web are determined by means of difference equations that are easily set up, and the sole mathematical device necessary for actual computation is the solution of simultaneous linear equations.

2. Deflections and Moments in Beams.—The method of calculation of beams presented in this section possesses no advantages over other simpler methods. It is intended solely as an introduction to the use of difference equations or differences in the study of flat plates. The purpose of this introductory treatment is to enable the extreme simplicity and ease of application of the method of the elastic web to be clearly explained. As a matter of fact, beams are limiting cases of certain types of plates and in consequence, results obtained for such plates should be valid for the corresponding limiting case beam.

We start from the well-known string polygon as applied to beams. Fig. 1 indicates a simple beam, uniformly loaded. If we divide the beam, as shown in the figure, into segments so that each is one-fourth the span of the beam (the two half segments nearest the support together being considered as one segment) and replace the uniform load in each by an equal concentrated load acting at the center of gravity of the uniform load, we can draw an equilibrium polygon. The two half segments at the support are omitted, for simplicity, and also because they do not appreciably alter the results. This equilibrium polygon has the same shape that would be assumed by a string loaded with the loads P and restrained by a force at the supports, whose horizontal component is equal to the pole distance H, of the associated force polygon. As we increase the number of divisions of our beam indefinitely, the string polygon approaches the condition of a cable loaded with a load uniformly distributed along its horizontal projection, and it therefore hangs in a parabolic arc. The moment at any point is equal to the intercept between the horizontal and the string polygon, multiplied by the pole distance H.[1] If we make

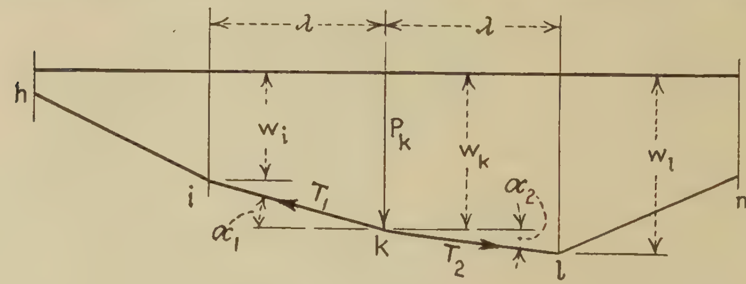

FIG. 2.—ANALYTICAL RELATIONSHIPS OF EQUILIBRIUM POLYGON.

H equal to unity, the cable forms a moment diagram for the uniformly loaded beam, and the string polygon becomes a moment diagram in the case in which concentrated loads are substituted for the uniform loads.

It is not necessary to use graphical construction to find the deflections of the string polygon. Consider the equilibrium conditions at one vertex of the polygon. Fig. 2 represents a portion of the equilibrium polygon whose associated force polygon has a pole distance of unity. The load P_k is applied at k; w_k is the deflection of k; w_i and w_l are deflections of the adjoining points i and l. The vertices of the polygon are assumed to be at equal horizontal distances, λ, from each other.

For equilibrium in the vertical and in the horizontal directions,

$$P_k = T_1 \sin \alpha_1 - T_2 \sin \alpha_2 \qquad (1)$$

$$T_1 \cos \alpha_1 = T_2 \cos \alpha_2 = H \qquad (2)$$

Dividing (1) by H or the corresponding value in (2),

$$\frac{P_k}{H} = \tan \alpha_1 - \tan \alpha_2 \qquad (3)$$

[1] Johnson, Bryan and Turneaure, Modern Framed Structures, Part I. (Ninth Edition) Page 43 et seq.

But, $\tan \alpha_1 = \frac{w_k - w_i}{\lambda}$; $\tan \alpha_2 = \frac{w_l - w_k}{\lambda}$

Substituting in (3),

$$\frac{P_k}{H} = \frac{2w_k - w_i - w_l}{\lambda}$$

If $H = 1$

$$2w_k - w_i - w_l = P_k \cdot \lambda$$

If p is the value of the uniformly distributed load, $P_k = p \cdot \lambda$ and

$$2w_k - w_i - w_l = p\,\lambda^2 \qquad (4)$$

Equation (4) forms the basis for the analysis of the beam, and is therefore an important equation.[1] Let us apply it to the beam of Fig. 1. Here $\lambda = \frac{l}{4}$. At the supports, w is zero, since the moment is zero. First apply equation (4) to point 1.

$$2\,w_1 - w_2 = p\,\lambda^2 \qquad (5)$$

From symmetry, $w_3 = w_1$. We now apply equation (4) to point 2.

$$2\,w_2 - 2\,w_1 = p\,\lambda^2 \qquad (6)$$

Solving simultaneous equations (5) and (6),

$$w_2 = 2\,p\,\lambda^2 \qquad w_1 = \tfrac{3}{2}\,p\,\lambda^2$$

Since $\lambda = \frac{l}{4}$,

$$w_1 = \tfrac{3}{32}\,p\,l^2 \qquad w_2 = \tfrac{1}{8}\,p\,l^2$$

H was made unity, consequently these ordinates should be numerically equal to the bending moment at the corresponding points. It is readily ascertained that both values are correct.

A well known theorem[2] in the strength of materials states that the deflection of any point of a beam is equal numerically to the moment produced at that point, by loading the beam with an imaginary load equal to the $\frac{M}{E\,I}$ diagram. In our case M_k is numerically equal to w_k, therefore we load the beam with the $\frac{w}{E\,I}$ diagram. For any point k, w_k is assumed as uniformly distributed over the space λ and E and I are considered constant. Then the load at each point k will be $\frac{w_k\lambda}{E\,I}$. Let us draw a new string polygon with H equal to unity, for this loading and call its ordinate at k, z_k. Equation (4) then becomes

$$2\,z_k - z_i - z_l = \frac{w_k\lambda^2}{E\,I} \qquad (7)^3$$

[1] This equation can be written $(w_k - w_i) - (w_l - w_k) = p\lambda^2$. The difference operator Δ is defined as $(\Delta w_k)_x = w_l - w_k$, (taken along the x axis) consequently equation (4) can be written $\Delta(\Delta\, w_k)_x$ (or second difference of w_k) $= (\Delta^2\, w_k)_x = -p\lambda^2$.

[2] Parcel and Maney, Statically Indeterminate Stresses, p. 52. Swain, Structural Engineering, Strength of Materials, p. 223.

[3] Equation (7) can be written $(\Delta^2 z_k)_x = -\frac{w_k\lambda^2}{E\,I}$

This is the fundamental equation relating the deflections of the second string polygon and the beam bending moments. z_k is numerically equal to the beam deflection at k. Apply this equation to our beam, first at point 1, then at point 2. Note that $z_3 = z_1$. At the supports, $z = 0$.

$$2\,z_1 - z_2 = w_2\,\lambda^2 = \tfrac{3}{2}\,p\,\lambda^4$$
$$-2\,z_1 + 2\,z_2 = w_2\,\lambda^2 = 2\,p\,\lambda^4$$

Solving, $z_2 = \frac{7}{2}\,p\,\lambda^4 = \frac{7}{512}\,p\,l^4 = .0137\,p\,l^4$

$z_1 = \frac{5}{2}\,p\,\lambda^4 = \frac{5}{512}\,p\,l^4 = .00978\,p\,l^4$

The correct values are $z_1 = .00928\,p\,l^4$, and $z_2 = .01302\,p\,l^4$, or about 5.4 per cent less in each case. With more divisions, greater accuracy can be obtained, in fact 8 divisions give results that vary about 1.2 per cent from the correct values.

If we substitute the values for w_k, w_i and w_l as obtained from equation (7) in equation (4), (point h being to left of i and point m to right of l)

$$\frac{E\,I}{\lambda^2}\,[2\,(2\,z_k - z_i - z_l) - (2\,z_i - z_h - z_k) - (2\,z_l - z_k - z_m)] = p\,\lambda^2$$

or $$z_h - 4\,z_i + 6\,z_k - 4\,z_l + z_m = \frac{P_k\lambda^2}{E\,I} = \frac{p\,\lambda^4}{E\,I} \quad \text{(8)[2]}$$

This equation expresses the ordinates of the second string polygon and in consequence, the deflections of the beam, in terms of the loading, on the beam. It can be applied directly to the beam. As before, $z = 0$ at the supports. When we apply equation (8), however, to point 1, there will be required a term for an imaginary point 1′, a distance λ to the left of the left support. This can be obtained from the condition that at the support $w = 0$ and applying equation (7) there, $-z_1' - z_1 = 0$, or $z_1' = -z_1$. For point 1,

$$-z_1 + 6\,z_1 - 4\,z_2 + z_1 = 6\,z_1 - 4\,z_2 = \frac{p\,\lambda^4}{E\,I}$$

For point 2,

$$-4\,z_1 + 6\,z_2 - 4\,z_1 = -8\,z_1 + 6\,z_2 = \frac{p\,\lambda^4}{E\,I}$$

Solving, we get, $z_1 = \frac{5}{2}\frac{p\,\lambda^4}{E\,I}$, $z_2 = \frac{7}{2}\frac{p\,\lambda^4}{E\,I}$ as before.

For partial, or non-uniformly distributed, or concentrated loads, p is modified in a perfectly obvious manner, the load P_k at every vertex representing the total load for a distance $\frac{\lambda}{2}$ on each side of k. For fixed or continuous beams, the same equations apply, except that the end conditions differ. In these cases, generally, the moments will not be known at the ends or supports, and it becomes necessary to use equation (8) first, finding the deflections. Moments are then found by substituting the values of the deflections in equation (7).

[2] Equation (8) can be written $\Delta^2(\Delta^2\,zk)x = (\Delta^4\,zk)x = -\frac{p\,\lambda^4}{E\,I}$.

3. The basic equation for solution of flat plates.—The differential equation of the beam derived from the common theory of flexure[1] is

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \quad \dots\dots (9)$$

y being the deflection of any point whose position is given by x, and M being the bending moment at that point. Since

$$\frac{dM}{dx} = V \text{ and } \frac{dV}{dx} = p = \frac{d^2M}{dx^2}$$

where V is the shear and p is the intensity of loading at the same point,

$$\frac{d^4y}{dx^4} = \frac{p}{EI} \quad \dots\dots (10)$$

It will be seen later that these are analagous to the differential equations of plates. However, a plate cannot be treated satisfactorily as two sets of superimposed beams at right angles. Two such imaginary sets of beams would not have independent deflections and the resistances at the edges of the elementary beams could not be neglected without serious error. In addition, the lateral deformation produced by the fiber stresses influences the resulting stresses and deflections considerably.

The basic differential equation[2] for flat plates, corresponding to equation (10) for beams, is

$$\frac{\delta^4\zeta}{\delta x^4} + 2\frac{\delta^4\zeta}{\delta x^2 \delta y^2} + \frac{\delta^4\zeta}{\delta y^4} = \frac{p}{N} \quad \dots\dots (11)$$

where

$$N = \frac{m^2}{m^2 - 1} \cdot \frac{Eh^3}{12}$$

ζ = deflection of plate at any point x, y.
p = intensity of loading at the same point.
m = Poisson's number (reciprocal of Poisson's ratio).
E = modulus of elasticity.
h = thickness of plate.

Equation (11) is derived upon the principal assumptions, (a) that the plate is medium-thick (i. e. not so thin that it approaches a membrane in action nor so thick that the distribution of stresses at the ends appreciably influences the results), (b) that the material is homogeneous, isotropic and perfectly elastic, (c) that a straight line perpendicular to the central surface of the plate before flexure remains straight and perpendicular to that surface after flexure, (d) that stress is proportional to strain.

Equation (11) can be resolved into simpler form by the use of the operator symbol

$$\nabla^2 = \frac{\delta^2}{\delta x^2} + \frac{\delta^2}{\delta y^2}$$

[1] James E. Boyd, Strength of Materials, (McGraw-Hill, 1924) p. 145.

[2] This non-homogeneous, fourth order partial differential equation is known as Lagrange's equation because Lagrange first derived it in 1813. No general solution for it has yet been found. In 1907 the French Academy of Science designated it as the prize problem for the Prix Vaillant. Special and particular solutions are known, of course.

Then

$$\frac{\delta^4\zeta}{\delta x^4} + 2\frac{\delta^4\zeta}{\delta x^2 \delta y^2} + \frac{\delta^4\zeta}{\delta y^4} = \left(\frac{\delta^2}{\delta x^2} + \frac{\delta^2}{\delta y^2}\right)\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right) =$$

$$\nabla^2\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right) = \nabla^2\nabla^2\zeta = \nabla^4\zeta = \frac{p}{N} \quad \ldots\ldots (12)$$

If we define a new function $\overline{M}$ (not having the same meaning as in beams)

$$-N\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right) = \overline{M} \quad \ldots\ldots (13)$$

the equation of the elastic surface becomes

$$\nabla^2 \overline{M} = -p \quad \ldots\ldots (14)$$

The function $\overline{M}$ is a measure of the bending of the plate. In Fig. 3 a portion of the plate is represented, dx wide, dy long and the thickness of the plate, h, thick.

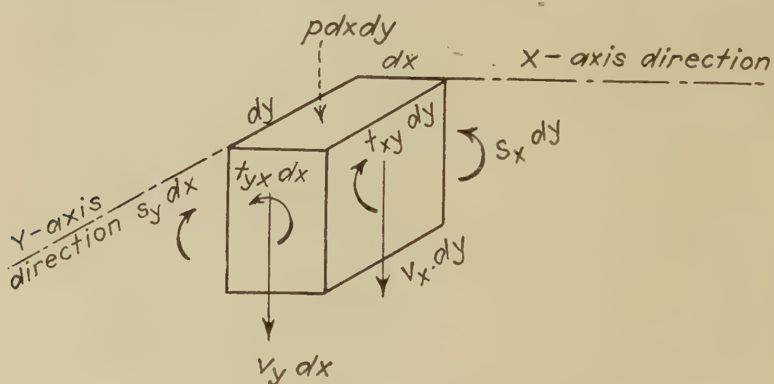

FIG. 3.—STRESSES ON ELEMENT OF PLATE.

s_x and s_y are bending moments normal to the faces of the prism, and are the resultants of the fiber stresses, v_x and v_y are vertical shearing stresses, and t_{xy} and t_{yx} are torsional stresses produced by the horizontal shears. All of these are in terms of one unit of width of element. It can be shown that,[1]

$$\left.\begin{aligned}
s_x &= -N\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{1}{m}\frac{\delta^2\zeta}{\delta y^2}\right)\\
s_y &= -N\left(\frac{\delta^2\zeta}{\delta y^2} + \frac{1}{m}\frac{\delta^2\zeta}{\delta x^2}\right)\\
t_{xy} &= -N\cdot\frac{m-1}{m}\,\frac{\delta^2\zeta}{\delta x\delta y}\\
v_x &= -N\cdot\frac{\delta}{\delta x}\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right)\\
v_y &= -N\cdot\frac{\delta}{\delta y}\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right)
\end{aligned}\right\} \quad \ldots\ldots (15)$$

[1] Dr. Ing. H. Marcus, Die Theorie elastischer Gewebe, Sec. 1.

$$s_x + s_y = -N \cdot \frac{m+1}{m}\left(\frac{\delta^2\zeta}{\delta x^2} + \frac{\delta^2\zeta}{\delta y^2}\right) = \frac{m+1}{m}\overline{M}$$

A membrane or thin plate subjected to fiber stresses parallel to the surface only, is described by the differential equation[1]

$$\nabla^2 w = \frac{\delta^2 w}{\delta x^2} + \frac{\delta^2 w}{\delta y^2} = -\frac{p}{S} \quad \text{.......(16)}$$

where w is the deflection of the plate at any point x, y; p is the intensity of loading, and S is the horizontal component of the surface stresses. From the conditions of static equilibrium, S is constant for the entire membrane. If S is made equal to unity, then comparing equation (16) with equation (14), we can state the following law: *A membrane loaded with the same loading, p, and having a value of $S = 1$, forms a moment diagram for the elastic plate.* Note that the "moment" in the above statement is really the moment-sum function $\overline{M}$, defined as above. In a manner perfectly analagous to that used for beams it can be similarly shown that the following law is true: *A membrane loaded with "elastic" loads $p_1 = \dfrac{\overline{M}}{N}$ and having a value of $S = 1$, forms a deflection diagram of the elastic plate.*

Differentiating equation (13) we obtain,

$$\frac{\delta^2\overline{M}}{\delta x^2} + \frac{1}{m}\frac{\delta^2\overline{M}}{\delta y^2} = -N\left(\frac{\delta^4\zeta}{\delta x^4} + \frac{m+1}{m}\frac{\delta^4\zeta}{\delta x^2\delta y^2} + \frac{1}{m}\frac{\delta^4\zeta}{\delta y^4}\right)$$

$$\frac{\delta^2\overline{M}}{\delta y^2} + \frac{1}{m}\frac{\delta^2\overline{M}}{\delta x^2} = -N\left(\frac{\delta^4\zeta}{\delta y^4} + \frac{m+1}{m}\frac{\delta^4\zeta}{\delta x^2\delta y^2} + \frac{1}{m}\frac{\delta^4\zeta}{\delta x^2}\right)$$

Differentiating equations (15), we get

$$\nabla^2 s_x = \frac{\delta^2 s_x}{\delta x^2} + \frac{\delta^2 s_x}{\delta y^2} = -N\left(\frac{\delta^4\zeta}{\delta x^4} + \frac{m+1}{m}\frac{\delta^4\zeta}{\delta x^2\delta y^2} + \frac{1}{m}\frac{\delta^4\zeta}{\delta y^4}\right)$$

$$\nabla^2 s_y = \frac{\delta^2 s_y}{\delta x^2} + \frac{\delta^2 s_y}{\delta y^2} = -N\left(\frac{\delta^4\zeta}{\delta y^4} + \frac{m+1}{m}\frac{\delta^4 x}{\delta x^2\delta y^2} + \frac{1}{m}\frac{\delta^4\zeta}{\delta x^4}\right)$$

If we define

$$\left.\begin{aligned} -\pi_x &= \frac{\delta^2\overline{M}}{\delta x^2} + \frac{1}{m}\frac{\delta^2\overline{M}}{\delta y^2} \\ -\pi_y &= \frac{\delta^2\overline{M}}{\delta y^2} + \frac{1}{m}\frac{\delta^2\overline{M}}{\delta x^2} \\ -\pi_{xy} &= \frac{m-1}{m}\frac{\delta^2\overline{M}}{\delta x\delta y} \end{aligned}\right\} \quad \text{.......(17)}$$

then, combining these with the previous equations, we get,

$$\left.\begin{aligned} \nabla^2 s_x &= -\pi_x \\ \nabla^2 s_y &= -\pi_y \\ \nabla^2 t_{xy} &= -\pi_{xy} \end{aligned}\right\} \quad \text{.......(18)}$$

[1] See A. Föppl, Vorlesungen über technische Mechanik, Vol. V, sec. 30; also, A. and L. Föppl, Drang und Zwang, Vol. 1, sec. 35.

The law that can be derived from these last equations reads: *The membrane carrying loads defined by equations (17) and having $S = 1$, forms a moment diagram for the s_x, s_y, and t_{xy} moments of the elastic plate.*

The shearing forces v_x and v_y can be obtained directly by a simple transformation of the last two equations (15).

$$\left.\begin{aligned} v_x &= -N\frac{\delta}{dx}\nabla^2\zeta = \frac{\delta\overline{M}}{\delta x} \\ v_y &= -N\frac{\delta}{\delta y}\nabla^2\zeta = \frac{\delta\overline{M}}{\delta y} \end{aligned}\right\} \qquad (19)$$

From the preceding results, it is readily seen that the complete solution of the plate is made dependent upon the two differential equations,

$$\nabla^2\overline{M} = -p, \ \nabla^2\zeta = -\frac{\overline{M}}{N}$$

In plates that are freely supported at the edges, ζ and $\overline{M}$ are zero at these edges, and consequently, the first differential equation suffices to determine $\overline{M}$ for the plate, such plates are therefore called "Statically Determinate" to distinguish them from plates requiring the use of both differential equations for the determination of $\overline{M}$, such as plates fixed at the edges, which may be called "Statically Indeterminate." In the case of the statically determinate plates, the $\overline{M}$ diagram obtained from the membrane is a true moment diagram, but in the case of statically indeterminate plates, the $\overline{M}$ diagram can be obtained from the membrane in a manner analagous to the moment diagram for beams with restrained ends, in which a closing line for the equilibrium polygon must be found to give a true moment diagram. Therefore in the laws as stated above, the "moment diagram" means the form of that diagram, and in the case of statically indeterminate plates, the true position of the diagram with respect to the axes must also be found.

The solution of the plate has been made dependent upon the solution of the membrane and now the membrane will be replaced by the elastic web, that approximates the state of stress and strain in the membrane.

4. The Elastic Web with rectangular meshes.—Figure 4 represents an elastic web composed of two sets of perfectly elastic wires crossing at right angles. The mesh width is uniform in each direction, λ_x in the direction of the x axis and λ_y in the direction of the y axis. The nodes or intersection points of the web are designated as shown in the figure. The uniform load p is replaced by concentrated loads at each node equal to the total uniform load on the area tributary to that node, as is indicated for point k by the shading. The separate wires have direct tensile stresses R_i, R_l, R_m, and R_n respectively for the wires running from k to i, l, m, and n. ω_i and ω_l designate the angles that wires R_i and R_l make with the x axis and ω_m and ω_n the angles wires R_m and R_n make with the y axis.

From the conditions of statical equilibrium at k in the three directions, the following equations are obtained:

$$R_i \cos\omega_i - R_l \cos\omega_l = 0$$
$$R_m \cos\omega_m - R_n \cos\omega_n = 0$$
$$P_k + (R_l \sin\omega_l - R_i \sin\omega_i) + (R_n \sin\omega_n - R_m \sin\omega_m) = 0$$

Call H_x and H_y the horizontal components of stress in the wires running in the x and y directions respectively. Then the following relations hold:

$$R_i \cos \omega_i = R_l \cos \omega_l = H_x$$
$$R_m \cos \omega_m = R_n \cos \omega_n = H_y$$

Substituting in the previous equation,

$$H_x (\tan \omega_i - \tan \omega_l) + H_y (\tan \omega_m - \tan \omega_n) = P_k$$

By the definition of difference operator given in the footnote on page 4,

$$\lambda_x \tan \omega_l = w_l - w_k = (\Delta w_k)_x$$
$$\lambda_y \tan \omega_n = w_n - w_k = (\Delta w_k)_y$$
$$\lambda_x (\tan \omega_l - \tan \omega_i) = (w_l - w_k) - (w_k - w_i) = \Delta_x (\Delta w_k)_x = (\Delta^2 w_k)_x$$
$$\lambda_y (\tan \omega_n - \tan \omega_m) = (w_n - w_k) - (w_k - w_m) = \Delta_y (\Delta w_k)_y = (\Delta^2 w_k)_y$$

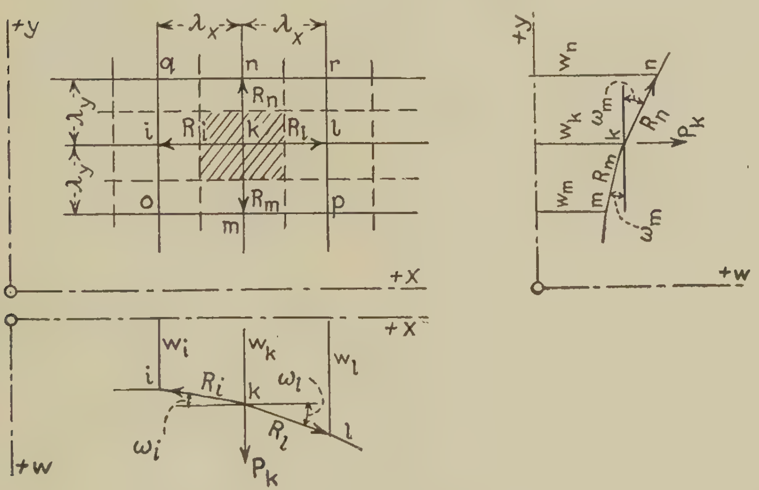

FIG. 4.—ELASTIC WEB RECTANGULAR MESHES.

Substituting above,

$$\frac{H_x}{\lambda_x} (\Delta^2 w_k)_x + \frac{H_y}{\lambda_y} (\Delta\, w_k)_y = -P_k \quad \text{.......} (20)$$

We now relate this to the membrane by noting that from the definition for S given on page 8,

$$H_x = \lambda_y S_x,\ H_y = \lambda_x S_y.$$

Also,

$$P_k = p_k \lambda_x \lambda_y$$

Consequently,

$$\frac{S_x}{\lambda^2_x} (\Delta^2 w_k)_x + \frac{S_y}{\lambda^2_y} (\Delta^2 w_k)_y = -p_k \quad \text{.......} (21)$$

If $H_x = H_y = H$, or $S_x = S_y = S$

$$\frac{1}{\lambda_x}\,(\Delta^2 w_k)_x + \frac{1}{\lambda_y}\,(\Delta^2 w_k)_y = -\frac{P_k}{H} \quad\quad (22)$$

$$\frac{(\Delta^2 w_k)_x}{\lambda^2_x} + \frac{(\Delta^2 w_k)_y}{\lambda^2_y} = -\frac{p_k}{S} \quad\quad (23)$$

If $\lambda_x = \lambda_y = \lambda$ we obtain the simple formula,

$$(\Delta^2 w_k)_x + (\Delta^2 w_k)_y = -P_k \,.\, \frac{\lambda}{H} = -p_k \,.\, \frac{\lambda^2}{S}$$

This can also be written,

$$4\,w_k - (w_i + w_l + w_m + w_n) = \frac{\lambda}{H}\,P_k = \frac{\lambda^2 p_k}{S} \quad\quad (24)$$

This is the basic equation used in the solution of square plates, freely supported at the edges.

Equation (24) for the plate is analagous to equation (4) for the beam. Note also that if the mesh widths are decreased indefinitely, equation (23) in the limit becomes the basic equation of the membrane,

$$\frac{\delta^2 w}{\delta x^2} + \frac{\delta^2 w}{\delta y^2} = -\frac{p}{S}$$

just as equation (4) in the limit becomes the equation of the cable,

$$\frac{d^2 M}{dx^2} = -p$$

For rectangular plates, whose ratio of longer side to shorter side is

$$\frac{\lambda_x}{\lambda_y} = \kappa$$

equation (23) becomes

$$(\Delta^2 w_k)_x + \kappa^2\,(\Delta^2 w_k)_y = -p_k \,.\, \frac{\lambda^2_x}{S}$$

or in expanded form

$$2\,w_k\,(1 + \kappa^2) - (w_i + w_l) - \kappa^2\,(w_m + w_n) = p_k \,.\, \frac{\lambda^2_x}{S} = \kappa^2 p_k \,.\, \frac{\lambda^2_y}{S} \quad\quad (25)$$

This is the basic equation for freely supported rectangular plates.

To determine the deflections, the same equations are used, only the load p_k becomes the "elastic load" $\frac{\overline{M}.}{N}$ The value of S can be made unity, and then the values of w will numerically equal the moment-sum $\overline{M}$ at the corresponding points. The values of ζ will be numerically equal to the values of $\frac{z}{N}$ at the corresponding points. Note that in applying equations (24) and (25) in finding deflections, w is replaced by z. Having the values of $\overline{M}$ and ζ we can find the stress-moments, torsions and shears directly from them.

Loading our web now with the elastic weight $p_k = w_k$ and taking S equal to unity, using equation (23),

$$\frac{z_i - 2 z_k + z_l}{\lambda^2_x} + \frac{z_m - 2 z_k + z_n}{\lambda^2_y} = \frac{1}{\lambda^2_x}(\Delta^2 z_k)_x + \frac{1}{\lambda^2_y}(\Delta^2 z_k)_y = - w_k = -\overline{M}_k. \quad (26)$$

We now evaluate the various partial derivatives necessary for determining the stress-moments, torsions and shears.

$$\left.\begin{aligned}
\left(\frac{\delta z}{\delta x_k}\right) &= \frac{z_l - z_i}{2\delta_x} = \frac{(\Delta z_k)_x}{\lambda_x} \\
\left(\frac{\delta z}{\delta y_k}\right) &= \frac{z_n - z_m}{2\lambda_y} = \frac{(\Delta z_k)_y}{\lambda_y} \\
\left(\frac{\delta^2 z}{\delta x^2_k}\right) &= \frac{z_i - 2 z_k + z_l}{\lambda^2_x} = \frac{1}{\lambda^2_x}(\Delta^2 z_k)_x \\
\left(\frac{\delta^2 z}{\delta y^2_k}\right) &= \frac{z_m - 2 z_k + z_n}{\lambda^2_y} = \frac{1}{\lambda^2_y}(\Delta^2 z_k)_y \\
\left(\frac{\delta^2 z}{\delta x \delta y_k}\right) &= \frac{(z_o + z_r) - (z_p + z_q)}{4\lambda_x\lambda_y} = \frac{(\Delta^2 z_k)_{xy}}{\lambda_x\lambda_y}
\end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Substituting these values in equations (15),

$$\left.\begin{aligned}
s_x &= -\left(\frac{\delta^2 z}{\delta x^2} + \frac{1}{m}\frac{\delta^2 z}{\delta y^2}\right) = \frac{2 z_k - z_i - z_l}{\lambda^2_x} + \frac{1}{m}\frac{2 z_k - z_m - z_n}{\lambda^2_y} \\
s_y &= -\left(\frac{\delta^2 z}{\delta y^2} + \frac{1}{m}\frac{\delta^2 z}{\delta x^2}\right) = \frac{2 z_k - z_m - z_n}{\lambda^2_y} + \frac{1}{m}\frac{2z_k - z_i - z_l}{\lambda^2_x} \\
t_{xy} &= -\frac{m-1}{m} \cdot \frac{\delta^2 z}{\delta x \delta y} = \frac{m-1}{m}\left[\frac{(z_p + z_o) - (z_o + z_r)}{4\lambda_x\lambda_y}\right] \\
v_x &= \frac{\delta w}{\delta x} = \frac{1}{2\lambda_x}(w_l - w_i) \\
v_y &= \frac{\delta w}{\delta y} = \frac{1}{2\lambda_y}(w_n - w_m)
\end{aligned}\right\} (28)$$

Substituting the values of ζ and $\overline{M}$ for z and w,

$$\frac{\lambda^2_x}{N} s_x = (2\zeta_k - \zeta_i - \zeta_l) + \frac{1}{m}\kappa^2(2\zeta_k - \zeta_m - \zeta_n)$$

$$\frac{\lambda^2_x}{N} s_y = \kappa^2(2\zeta_k - \zeta_m - \zeta_n) + \frac{1}{m}(2\zeta_k - \zeta_i - \zeta_l)$$

$$\frac{\lambda^2_x}{N} t_{xy} = \frac{m-1}{4m}\kappa[(\zeta_p + \zeta_q) - (\zeta_o + \zeta_r)]$$

$$2\lambda_x\lambda_x = \overline{M}_l - \overline{M}_i$$

$$2\lambda_x\lambda_y = \kappa(\overline{M}_n - \overline{M}_m)$$

The value of m that is used in the above formulas influences the resulting values to a considerable extent. For concrete, at the usual values of working stresses, m is approximately 5. However, as the stresses approach the ultimate, m increases for concrete in tension[1] and as the most unfavorable case would be m equal to infinity, that value may be considered as the proper one to use in determining stresses, etc., in plates approaching a condition of rupture. In determining the value of working stress to be used in the plate. if m is taken as infinity, no notice need be taken of the fact that as the stresses in the plate increase toward the ultimate strength of the materials, the moments do not change proportionately to the loads, because the stresses will have been calculated upon the basis of the most unfavorable condition of the plate. If a value of $m = 5$ is chosen, however, the working stresses may be taken

FIG. 5.—SQUARE PLATE, WITH 16-MESH ELASTIC WEB.

less conservatively to allow for the less rapid increase of stresses that occur as the loads increase. The maximum fiber stress at any point in the plate is given by[2]

$$s_{\substack{max. \\ min.}} = \frac{1}{2}\left[(s_x + s_y) \pm \sqrt{(s_x - s_y)^2 + 4\,(t_{xy})^2}\right]$$

5. Application to the solution of square plates.—Figure 5 represents a square plate freely supported at the four edges. "Freely supported" is taken to mean that the corners are so held that they cannot lift from the supports. Dr. Marcus has evaluated this lifting force and shown that it is twice the value of the torsion at the corners.[3] The plate will be divided into 16 squares as shown, each line within the square representing a wire of the elastic web. First a uniformly distributed load will be assumed. For a square plate

[1] This is indicated from experiments by von Rudeloff, see H. 5 u. 21 Veröffentlichungen des Deutshen Ausschusses fur Eisenbeton.

[2] Dr. Ing. H. Marcus, Die Theorie elastischer Gewebe, sec. 4, p. 37.

[3] Dr. Ing. H. Marcus, Die Theorie elastischer Gewebe, sec. 6, p. 41.

$k = 1$. In this case $\lambda = \frac{l}{4}$. Applying equation (24) to each point in succession and noting the symmetry indicated by the numbering of the plate,

$$\begin{aligned} 4\,w_1 - 2\,w_2 &= p\,\lambda^2 \\ -2\,w_1 + 4\,w_2 - w_3 &= p\,\lambda^2 \\ -4\,w_2 + 4\,w_3 &= p\,\lambda^2 \end{aligned}$$

Solving these three simultaneous linear equations,

$$\begin{aligned} \overline{M}_1 &= w_1 = .04297\,p\,l^2 \\ \overline{M}_2 &= w_2 = .05469\,p\,l^2 \\ \overline{M}_3 &= w_3 = .07031\,p\,l^2 \end{aligned}$$

Loading the plate with elastic loads w_1, w_2, w_3 at points 1, 2 and 3 respectively, we get, using equation (24) again,

$$\begin{aligned} 4\,z_1 - z_2 &= w_1\lambda^2 = .002686\,p\,l^4 \\ -2\,z_1 + 4\,z_2 - z_3 &= w_2\lambda^2 = .003418\,p\,l^4 \\ -4\,z_2 + 4\,z_3 &= w_3\lambda^2 = .004394\,p\,l^4 \end{aligned}$$

Solving these three simultaneous linear equations,

$$\zeta_1 = \frac{z_1}{N} = .002136\,\frac{p\,l^4}{N}$$

$$\zeta_2 = \frac{z_2}{N} = .002930\,\frac{p\,l^4}{N}$$

$$\zeta_3 = \frac{z_3}{N} = .004028\,\frac{p\,l^4}{N}$$

If a web with 64 meshes had been chosen in place of the 16-mesh web, the values of $\overline{M}$ would be changed at most by 4 per cent and the values of ζ by about ½ per cent. Thus it is evident that with a comparatively coarse mesh web, results of fair accuracy can be obtained with very little labor. A comparison of these results with those obtained by Nadai and Estanave (see footnote p. 1) indicate differences not to exceed 2½ per cent and in some cases not over $\frac{1}{10}$ per cent from their results when the 64 mesh web is used. Consequently it may be concluded that even the 16 mesh web gives fairly accurate results and the 64 mesh web produces results of sufficient accuracy for most practical purposes. This is true for $\overline{M}$ and ζ, but for the stress-moments, torsions and shears, the use of the web with closer meshes is necessary to give accurate results.

Using the values of M and ζ obtained from the 64 mesh web, for $m = 5$, $s_x = s_y = .04365\,p\,l^2$, and for m equal to infinity, $s_x = s_y = .036385\,p\,l^2$. The maximum shearing stress occurs at point IV and is $.3387\,p\,l$. The supporting reactions are

$$\begin{aligned} a_{\mathrm{I}} &= .2545\,p\,l \\ a_{\mathrm{II}} &= .3544\,p\,l \\ a_{\mathrm{III}} &= .4037\,p\,l \\ a_{\mathrm{IV}} &= .4289\,p\,l \end{aligned}$$

and the corner lifting force, $C = .0594\,p\,l^2$.

For concentrated forces, similarly, the values obtained show good agreement with other analyses. One difficulty arises, however. In the immediate neighborhood of the concentration the stresses increase indefinitely as the mesh width is decreased. As the load is concentrated upon consecutively smaller areas, the stresses increase without reference to the size of the plate. Since truly concentrated forces are not met with in nature, it becomes necessary to use a load distributed over a very small area, representing the actual conditions as nearly as possible. Similarly rectangular plates can be readily solved by these methods, without any difficulty. Tables showing the stresses, deflections, etc., of square and rectangular plates, with various uniform, concentrated and unsymmetrical loads are being prepared and it is hoped that at the next annual meeting a more complete discussion will be available, together with these tables.

While the assumptions upon which these results have been obtained are practically the same as are used for the design of reinforced concrete beams, one important difference appears. In the case of plates there exists sometimes a condition of reinforcing which is different in the two directions. The plate does not have a constant moment of inertia. Dr. Marcus has investigated this analytically and finds that the deflections are not appreciably influenced by even extremely large variations in moment of inertia. Even the stresses are not very greatly influenced, because in one case investigated, where there was a difference of moment of inertia in the two directions of 400 per cent, the fiber stresses differed only 22 per cent. So we may say that for most practical cases this effect can be neglected.

JOSEPH A. WISE (*By letter*).—I would like to add one or two statements to the paper to clarify certain concepts. The stress moments, s_x, s_y, are the proper ones to use in applying the usual rectangular reinforced-concrete beam formulas for design purposes in the same way that one uses the usual bending moments in beams. In regard to shearing stresses, it is not believed that the resultant diagonal tensile stress will be as high proportionately as in the case of the usual rectangular reinforced-concrete beam, and that higher allowable unit shearing stresses can be safely permitted in this type of flat slab. There also may be some question as to the effect of possible cracking of the concrete below the neutral axis upon the stiffness of the plate in various directions and also upon its torsional resistance. The first effect, I believe, can be neglected for most practical cases as it will be about the same in both directions. The second effect is extremely difficult to evaluate, but it is not believed that it will very greatly change the results. A reduction in the torsional resistance of the slab probably means an increase in the stress moments; but this is to some extent offset by the fact that when the concrete stress attains moderately high values a certain amount of plastic flow occurs with the redistribution of the stress, relieving to some extent the more highly stressed areas of part of their higher stress. It is probable that considering the two effects, that is, the reduced torsional strength and the phenomena of plastic flow, as offsetting one another, we can neglect both and merely consider the stress produced by the stress moments as above indicated. **Prof. Wise.**

Review of the Discussion of the Reinforced-Concrete Column.*

By Phil J. Markmann.**

The correctness of my analysis of "The Reinforced-Concrete Column," p. 127 to p. 166 of the 1927 *Proceedings*, is disputed by all contributors to the discussion, p. 167 to p. 178. The objections raised by four of these contributors, viz: Mr. Hadley, Mr. Lagaard, Mr. Richart and Prof. Talbot, are based on two allegations which will be stated presently. One other contribution is irrelevant, one other too vague for a comment. The two allegations made by the four of the contributors named are correlative in fact, and either *stand* or *fall* together.

The first *allegation* is:

The reinforced-concrete column after being contracted by the unit contractive force $\frac{E}{n} m$ of the concrete acting against the resistance of the steel to what H. M. Hadley in his diagram on p. 167 calls its "actual length," when subjected to any external load will, like any other column, be shortened by the said external load.

In answer to this first allegation my paper demonstrates the truth of *the following statement*, which, therefore, supersedes the above allegation: The said "actual length" begins to be shortened by the external load *only after* the said load begins to exceed $\frac{E}{n} m\ (1 - p)$, *and not before.*

The second *allegation* is:

The length of the concrete in the said "actual length" column must shorten while its "stress" increases from f_{cr} (tension) to f_{ct} (compression), and, therefore, the steel must shorten also.

In answer to this second allegation my paper demonstrates the truth of *the following statement*, which, therefore, supersedes the above allegation: During the whole time in which the concrete "stress" increases from f_{cr} to f_{ct} $\left[\text{as } \frac{P_e'}{A} \text{ increases from 0 to } \frac{E}{n} m\ (1 - p)\right]$ the "force" acting upon the concrete is f_{ct} (constant); therefore, this *constant* "force" f_{ct} causes the *constant* deformation $\frac{f_{ct}}{\frac{E}{n}} = m \frac{1 - p}{1 + (n - 1)\ p}$ of the concrete. At the same time the

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* This paper follows the author's paper and the attendant discussion appearing in the 1927 *Proceedings*, Vol. 23, p. 123.

** Consulting Engineer, St. Louis, Mo.

constant steel "stress" f_{st} causes the *constant* deformation $\frac{f_{st}}{E} = m \frac{1-p}{1+(n-1)p}$ of the steel. The deformation of the concrete being *identical* with that of the steel, and *constant* while $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m (1-p)$, the shear between the two does *not* change from what it is when $\frac{P_e'}{A} = 0$, viz. $f_{st} \times p = -f_{cr}(1-p)$.

While $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m (1-p)$ the "force" *acting upon the concrete* (f_{ct}) is $\frac{1}{n}$ of the "force" *acting upon the steel* (f_{st}), and, therefore, when the "force" f_{ct} *acting upon the concrete* becomes *identical* with the "stress" f_{ct} *of the concrete*, "elastic equilibrium" of the two *materials* is attained. This happens when $\frac{P_e'}{A} = \frac{E}{n} m (1-p)$.

The reinforced-concrete column is unique as a column, because in service, it has not only to resist the pressure by the external load, but the internal contractive force $\frac{E}{n} m$ of the concrete, as it hardens, develops normal stresses of the concrete and of the steel, as well as shear stresses along their contact area, as follows:

(1) a tensile stress f_{cr} in the concrete } *at the middle of the column.*
(2) a compressive stress f_{st} in the steel }
(3) a total shear $f_{cr}(1-p)$ acting *from each end* toward the mid-section of the column, *on the concrete side.*
(4) a total shear $f_{st} \times p$ equal to $f_{cr}(1-p)$, but acting in the opposite direction viz. *from the mid-section* toward each end, *on the steel side* of the contact area.

Referring to Mr. Hadley's diagram on p. 166, the "original length" of the column shortened to its "actual length" is the column with the action of the internal force completed. The steel and concrete in this column have "stresses" of opposite nature, the totality of the concrete stress balancing the totality of the steel stress [$f_{cr}(1-p) + f_{st} \times p = 0$].

Neither the steel nor the concrete in this "actual length" of column have their "free" lengths; the steel is shorter the concrete is longer than their respective "free" lengths. *Now, while a column composed of two constituent elements, and both neutral in stress prior to being subjected to an external axial compression, will immediately respond to the load by a strain increasing with the load, the ratio of their respective compressive stresses being the ratio of their respective moduli of elasticity, it should at least be suspected that the reinforced-concrete column of "actual length," with the steel and the concrete having stresses differing from neutral in opposite directions, will respond to the load in a different manner.*

I contend and am demonstrating that an external load $< \frac{E}{n} m (1 - p)$ in amount causes *no further* shortening, and that *only* the excess of the external load over the amount $\frac{E}{n} m (1 - p)$, viz. $\frac{P_e'}{A} - \frac{E}{n} m (1 - p)$, [the 2d part of the external load] *causes a further shortening,* as follows:

With no external load on the reinforced-concrete column the concrete is acted upon by the unit contractive force $\frac{E}{n} m$, which *would* shorten the concrete from its original length 1 to length $1 - m$. But it is also acted upon by the tension f_{cr} due to the resisting shear of the steel which tension *would* lengthen the concrete from its length $1 - m$ to length $1 - \epsilon$. The *resultant of these two forces* is $\frac{E}{n} m + f_{cr} = f_{ct}$; *it is this resultant force* which *actually* shortens the original length 1 to length $1 - \epsilon$. (See Fig. 3, p. 156.)

The full cohesive tension $-\frac{E}{n} m$ in the shrinking concrete (which is the full contractive force) is *reduced* by the pressure $\dfrac{\frac{P_e'}{A}}{1 - p}$ of an external load $\frac{P_c'}{A}$ on the column to the smaller tension $-\frac{E}{n} m + \dfrac{\frac{P_c'}{A}}{1 - p}$ and, therefore, $\frac{E}{n} m - \dfrac{\frac{P_c'}{A}}{1 - p}$ is now *the reduced contractive force.* The "stress" f_{cr} (the resultant of $-\frac{E}{n} m$ and f_{ct}) is *increased* by the external load pressure $\dfrac{\frac{P_e'}{A}}{1 - p}$ to $f_{cr} + \dfrac{\frac{P_c'}{A}}{1 - p}$.

Now when the external load equals $\frac{E}{n} m (1 - p)$ *the reduced contractive force* is $\frac{E}{n} m - \frac{E}{n} m = 0$, and *the increased "stress" of the concrete* is $f_{cr} + \frac{E}{n} m = f_{ct}$.

The *resultant* of this *reduced contractive force* and the *increased "stress" of the concrete* is $0 + f_{ct} = f_{ct}$.

The *resultant* f_{ct} of the two forces *acting upon the concrete* is *now* represented *solely* by *the compressive "stress"* f_{ct} *of the concrete,* because the contractive force in the concrete has been *reduced* to 0.

(The natural law governing the apportionment of the load to the concrete and to the steel, says: *none* of the load will go to the steel as long as the concrete "stress" is $< \frac{f_{st}}{n}$, and $\frac{f_{st}}{n} = f_{ct}$).

The external load $= \frac{E}{n} m(1-p)$ in amount, *transforms* the *resultant "force"* f_{ct} which *acts upon the concrete as long as the* external load is $< \frac{E}{n} m(1-p)$, into the *compressive "stress" f_{ct} of the concrete.* Therefore, the *force acting upon the concrete* does *not* change while $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m(1-p)$, and *with no change of force there is no change of deformation.*

These are the facts in the case, they show the two allegations to be *unwarranted assumptions.* All of the above is fully demonstrated on pp. 158 and 159 of the paper.

Prof. Talbot says, on p. 177, "Surely it cannot be true if one would stop to think that there be strains without stress," evidently referring to my showing the deformation of the concrete remaining *constant* while its "stress" changes (increases) from f_{cr} to f_{ct}, or from the residual tension f_{cr} to the compressive stress f_{ct}, which increase of "stress" occurs while the external load $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m(1-p)$ in amount.

Now, what is absolutely true is that *there cannot be a strain without a force, nor change in strain without a change in force,* and that *the strain remains constant* as long as *the force remains constant.* The fundamental definition of the modulus of elasticity of elastic bodies is

$$\text{Mod. of elast.} = \frac{\text{Unit force}}{\text{Unit strain}}$$

The *force* which causes the deformation (shrinkage) m of a *plain* concrete prism (the cohesion of the particles of the mass), $\frac{E}{n}$ being its modulus of elasticity, is $\frac{E}{n} m = -f_h$ (see eq. 4, p. 126). $\frac{E}{n} m$ is the *only force* acting upon *this* concrete. When the deformation m is accomplished, no further deformation takes place, it follows a force is then no longer exerted upon the concrete, and, therefore, the "stress" of the contracted (hardened) concrete = 0.

Now, in the *reinforced* concrete prism the same unit contractive force $\frac{E}{n} m = -f_h$ acts *in* the concrete *tending to shorten it;* opposed to this force, however, is the shear of the steel equivalent to a *unit force acting upon the concrete of* $-\frac{f_{st} \times p}{1-p} = f_{cr}$ (see pp. 127 and 153), *tending to lengthen* the concrete. The *resultant* of these two forces is $-f_h + f_{cr} = f_{ct}$ (see p. 127.) Therefore, *f_{ct} is the "unit force" in the equation of the modulus of elasticity of the concrete, while the "stress" of the concrete is f_{cr}, or between f_{cr} and f_{ct},* as $\frac{P_e'}{A}$ is either $= 0$ or $< \frac{E}{n} m(1-p)$.

Thus, with the hardening of the concrete completed and prior to the application of external load, the concrete "stress" is f_{cr}, the steel stress is f_{st}, the former a *tensile* and the latter a *compressive* stress (given by eqs. 8 & 7, resp., on p. 127, or eqs. b' and c', p. 157).

An external load $\frac{P_e'}{A} \leqq \frac{E}{n} m(1-p)$ in amount placed upon the column will *all be borne by the concrete area* $(1-p)$, because the concrete, prior to the application of external load and as long as $\frac{P_e'}{A}$ is not $\geqq \frac{E}{n} m(1-p)$ has a "stress" $< \frac{f_{st}}{n}$ or $< f_{ct}$.

The unit pressure by the external load $\frac{E}{n} m(1-p)$ upon the concrete is $\frac{E}{n} m = -f_h$, and this unit pressure added to the tensile stress f_{cr} gives $-f_h + f_{cr} = f_{ct}$ (see p. 127).

So the *resultant* of either the unit contractive force $\frac{E}{n} m$, or of the unit pressure $\frac{E}{n} m$ by the stated external load, and the "residual" stress f_{cr}, in either case, is f_{ct}. Furthermore, it has been shown on p. 159 that f_{ct} is the *resultant* force for any value of $\frac{P_e'}{A}$ between 0 and $\frac{E}{n} m(1-p)$.

Therefore, the unit force acting upon the concrete is *constant* while $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m(1-p)$, and the deformation ϵ of the concrete is

$$\epsilon = \frac{f_{ct}}{\frac{E}{n}} = m \frac{1-p}{1+(n-1)p} \text{ (see eq. } e', \text{ p. 157)}$$

Likewise, we have for the deformation of the steel under the *constant* compressive stress f_{st}, while $\frac{P_e'}{A}$ increases from 0 to $\frac{E}{n} m(1-p)$,

$$\epsilon = \frac{f_{st}}{E} = m \frac{1-p}{1+(n-1)p}$$

Thus the paradox of "strains without stress," or to be more precise, of *no change in strain with a change in "stress,"* is explained by the *simple fact* that, when $\frac{P_e'}{A} = 0$ and the concrete "stress" is f_{cr}, this "stress" f_{cr} is *not* the "unit force" acting upon the concrete, because there is the other concomitant force $\frac{E}{n} m = -f_h$ also acting upon the unit area of the concrete, the *resultant* of the two is f_{ct}, we may say:

$$\text{Mod. of elast.} = \frac{\text{Unit stress}}{\text{Unit strain}}$$

only when the unit "stress" is *the reaction to the "force" acting upon the unit*

area, but the "stress" f_{cr} when $\frac{P_e'}{A} = 0$, or the stress $> f_{cr}$ and $< f_{ct}$ when $\frac{P_e'}{A}$ is > 0 and $< \frac{E}{n} m (1 - p)$, *is not the reaction to the resultant force* f_{ct}, it is only *one* of its two components. When, however, the external load $\frac{P_e'}{A} = \frac{E}{n} m (1 - p)$ the "stress" of the concrete is *actually* f_{ct}, the "stress" *now* is *the reaction to the resultant force* f_{ct}. [When the external load $\frac{P_e'}{A}$ has reached the amount $\frac{E}{n} m (1 - p)$ or is $> \frac{E}{n} m (1 - p)$ the concrete and steel stresses are those given by eqs. 2 and 3 on p. 126.]

The above demonstration refutes all that Prof. Talbot has stated in his first ten lines on p. 177, from which he concludes: "The analysis has nothing to stand on."

No vital assertions other than the points made by Prof. Talbot have been made by any of the other contributors.

At the time I developed my equations for the internal concrete and steel stresses I was not aware that Matsumoto in Bulletin No. 126 of the Engineering Experiment Station, University of Illinois had "attempted" to do the same thing. He gives the "shrinkage" stresses of steel and concrete in his eqs. 5 and 6 on p. 21. Mr. Matsumoto has *also* overlooked the fact that two forces are acting concomitantly upon the concrete of the reinforced concrete column, as long as $\frac{P_e'}{A} < \frac{E}{n} m (1 - p)$.

On October 26, 1926, I apprised the Engineering Experiment Station of the University of Illinois of the error made by Matsumoto in developing his eqs. 5 and 6 on p. 21 of the said Bulletin.

When introducing my paper at the 1927 Convention of the Institute, I called attention to these equations giving results on an average 50 per cent too high. My introductory remarks were not published, but Prof. Talbot's concluding statement, on p. 178, refers to my comment on Matsumoto's work. He merely stated Bulletin 126 was not written by Mr. Richart but by Matsumoto, then at the University of Illinois, but did not commit himself on these equations which I had shown to be entirely out of line.

It should be said that the external load, or its first part, applied to a well-seasoned column is rather the exception in construction work. Ordinarily the gradual development of the internal concrete and steel stresses approximately coincides with the gradual application of external loads. This has been stated at the bottom of p. 138. As the hardening of the concrete begins and progresses the adhesion of the concrete to the steel is on a gradual increase, tensile stress in the concrete and compressive stress in the

steel begin and increase gradually, the increase in the steel compression is proportional to the increase of concrete tension, the steel compression reaches its maximum f_{st} *just as soon as the concrete is fully hardened.* The gradually and simultaneously developing concrete tension, however, is *more than neutralized* by the simultaneously and gradually applied pressure by the external load. Thus, while the steel stress has been increased to f_{st} *at completed hardening*, the resultant of the internal concrete stress f_{cr} and the pressure by the external load may be less, or $= f_{ct}$, according as the growth of the external load is slower, or just as fast, as the development of the internal shrinkage stresses. Therefore, when the loading of the column up to the amount $\frac{P_e'}{A} = \frac{E}{n} m (1 - p)$ proceeds at the same rate as the hardening of the concrete, the steel stress increasing gradually with the hardening of the concrete reaches its maximum f_{st} simultaneously with the concrete stress reaching the value f_{ct}; on the other hand, if the hardening is completed before as much external load as $\frac{E}{n} m (1 - p)$ is on the column, the steel stress f_{st} *remains constant from then on until the external load* $\frac{P_e'}{A}$ *equals* $\frac{E}{n} m (1 - p)$. The steel stress at the time when $\frac{P_e'}{A} = \frac{E}{n} m (1 - p)$ is on the column, provided the concrete has been fully hardened by or before that time, is f_{st}, and *the external load has in no-wise either caused or contributed to this stress, whether the concrete had been hardened prior to the application of the external load, or not.* When a progress in the deformation (shortening) of the column is observed during the application of the first part of the external load (the compensating load $\frac{P_e'}{A} = \frac{E}{n} m (1 - p)$, *this simply indicates that hardening of the concrete and contraction of it are still in progress, are not completed.* With the said load applied *to the hardened column, practically no increase in deformation should be expected until after the load begins to exceed the amount* $= \frac{E}{n} m (1 - p)$.

DISTRIBUTION OF BOND STRESS

The distribution of bond stress along the column as demonstrated on pp. 151 and 152 is not acceptable to Mr. Richart (p. 176). I stand pat on the *parabolic law* of shear distribution along the column, and *on the reasoning leading to it.* It is evident that the shear line must be a continuous curve from end to end of the column, with $x = 0$ and $\frac{dy}{dx} = \infty$, when $y = 0$.

The maximum bond stress at the ends with the parabola as the bond stress line is 1½ of what it would be by the straight line variation of shear. Prof. Talbot saying, on p. 178, "Matsumoto assumed a distribution as being perhaps the highest and as bringing the worst conditions," is in error, *it is the parabolic distribution that gives the greater maximum bond stress.*

POSTSCRIPT

There is only *one* amendment I wish in my own behalf and on my own motion, to make to my paper as now published. It is this:

The physical factor m has been defined, on p. 126, as "the unrestrained unit contraction of the completely hardened concrete." This means m is the unit contraction of a specimen made from the same concrete, but containing no steel. Now it is very reasonable to believe that some of this contraction occurs while the concrete is still in a plastic or near-plastic state, during which state its contracting movements are not opposed by the steel, even in case of deformed bars the concrete will readily recede at projecting shoulders. With the concrete still in a plastic or near-plastic state *no stress in either steel or concrete is developed.* Only the balance of the contraction—occurring after the concrete has become rigid—is effective in producing the actual *internal* stresses in concrete and steel. Calling m' the contraction occurring while the concrete is still plastic (this may be called the "slip"), then it is the balance $m - m'$ of the full free contraction that produces the internal stresses; therefore, m having been defined and understood in my paper as "the whole unrestrained contraction" it follows that the m in the eqs. 4, 5, 6, 7, 8, 9, 9a, 10, 11, 12, 12b, 12c and 12d should be replaced by $m \times \frac{m - m'}{m} = m - m'$, to give results comparable with actual test observations. The amount m' (the slip) of any one concrete for which the unrestrained contraction m has been measured, and for which $\frac{E_s}{E_c} = n$, and ϵ for a given steel ratio, have been found by tests, is given by the equation

$$m' = m - \epsilon \frac{1 + (n - 1)\,p}{1 - p}$$

and the ratio $\frac{m - m'}{m} = \frac{\epsilon}{m}\,\frac{1 + (n - 1)\,p}{1 - p}$, *for the given steel ratio.*

How a State Law Helped Concrete Building Units in Wisconsin.

By D. R. Collins.*

We, in Wisconsin, are mighty proud of the quality of concrete products throughout the state. We should make good concrete products for nature has endowed us with an abundance of splendid aggregate. But, for many years, we slipped along in the easiest manner possible. Perhaps the fact that there was such an abundance of natural aggregate was largely responsible for only a fair quality of product produced. In spite of all propaganda and educational matter published on the use of screened and cleaned materials for the manufacture of concrete products, most manufacturers were indifferent. What they were using seemed good enough to get by, and above all, the low price appealed to them.

Then came interest in a revision of the state building code relative to concrete products. Wisconsin was fortunate in having a state building code, but the section covering concrete masonry had been on the books for some time and was by many considered antiquated. News had filtered into the state about the almost incredible growth of the concrete building tile business in Detroit and of the development of cinder concrete in the eastern states. Under the Wisconsin code then existent the percentage of air space in concrete building units was limited to 33 per cent, and the absorption percentage was limited to 10 per cent of the dry weight. Those restrictions would not allow either concrete building tile or cinder units of any description to be used in public or industrial buildings covered by the state building code. That the use of the units was basically sound had already been proved in other localities—so why not change the code so that they might be used?

A meeting was called at the state capitol and presided over by the state building engineer in charge of enforcement of the code. Arguments were presented for the changes, but when the hearing was completed the building engineer gave the manufacturers to understand that any changes must be backed by their sincerity in seeing that the code was enforced. As the present code was still standing, it was the one with which they must comply. Concrete products were not in very high repute in Wisconsin at that time. Few men engaged in the business made any attempt to comply with the required tests and scarcely any took it upon themselves to see that the code was enforced.

Not many months after the meeting referred to, a small group formed the Wisconsin Concrete Products' Association with the avowed intention of bettering the quality of concrete products and at the same time securing

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a state building code that would allow concrete products to be used more extensively in the construction of public buildings and places of employment. Through the aid of the Portland Cement Association and the A. C. I. all data relative to the asked change in the code was collected and turned over to the state building engineer. But better still, this group of men started out in earnest to show their sincerity in code obedience and enforcement. There was a sudden rush of test specimens to the various laboratories in the state. The offices of the Portland Cement Association began to receive numerous inquiries on making better concrete. Many of the first tests showed building units far below the required standard and many of the subsequent tests showed units far above the standard. But by the second year of testing an astounding regularity appeared among the tests. The state code called for a compression of 700 lb. per sq. in. on gross area. These second-year tests showed on an average between 900 and 1,000 lb.—this excess being the safety factor the majority of manufacturers felt necessary. Testing laboratories that had tested perhaps half a dozen sets of block a year found it necessary to appoint a man to take charge of the increased testing.

This apparent sincerity on the part of the concrete products manufacturers was followed closely by the state building engineer. Copies of all tests, good or bad, were sent to his office. By this time his interest in concrete building units had come to the attention of the National Building Officials' Conference and he was designated as chairman of a committee to draw up a model code covering the units for submission to that body. This served to intensify his interest, but put an added burden on the group in Wisconsin whom he could watch at close range.

Finally the code was written and submitted for criticism to the group of Wisconsin concrete products men who had striven so hard to bring it about. A few rough spots were ironed out and the code adopted by the Wisconsin Industrial Commission. During all of the time the code was under discussion the manufacturers had been careful to see that their progress in the art of making quality concrete products was constantly brought to the attention of architects and contractors, and when the hampering limitations were lifted they were in a position to go after this class of business.

The new code made no mention of air space or void content, and the paragraphs relative to absorption made possible the use of cinders, slag, or Haydite as aggregates. It was not long before a lighter weight unit began to appear on the Wisconsin market. Where light weight aggregate was not available, concrete building units with thinner walls and a greater void or air space content appeared. Concrete building tile put in an appearance.

More air space in the units called for better concrete and better concrete called for better manufacturing methods. The little group of pioneers who started the movement for a new code were ready for this and willing to pass on the results of their experiences. It was surprising the number

of so-called small plants who now rejected the bank run materials they had been using and demanded screened and cleaned materials. Sand and stone companies furnishing washed materials were forced to create grades suitable for concrete products manufacture. Conversation about plants suddenly became more intelligent. "Fineness modulus," "water-cement ratio," "time of mix" suddenly became household words with even the average manufacturer. All through the state manufacturers set to it to produce a quality concrete. Many were greatly surprised at their reduction in breakage and the real savings that could be effected by using brains in making concrete products.

Testing laboratories again became swamped with test specimens. Where they had thought their limit had been reached in testing this class of material, they found the amount of it doubled. Quality concrete products had arrived in Wisconsin.

Best of all, what was happening became known generally among architects and builders. They now had confidence in a material they had thought suitable only for basements under small frame dwellings. More and more concrete building units were used in high grade work. In a few short years the concrete building unit rose from the position of a despised material to that of an accepted article.

One great and lasting good that interest in securing the new code has brought about is a welding together of manufacturers operating in all sections of the state. They are operating under a common law and meeting common requirements. They have common problems and are working them out together. Local codes are being changed to comply with the state code. They have achieved a unity of purpose.

And all the while these Wisconsin concrete products manufacturers are carefully studying better manufacturing methods so that they may economically improve their product, that they may always meet the Wisconsin building code and at the same time meet competition from other industries on an equal basis.

Discussion.—Law and Building Units.

C. L. Bourne.—I would like some information as to the value of the state tests in Wisconsin in the promotion and sale of concrete products. Mr. Bourne.

D. R. Collins.—The concrete product based on the state code has an almost inestimable value for advertising purposes. The state building engineer in writing the code placed a stamp of approval on concrete products made in compliance with it. The Wisconsin Concrete Products Association also issues a certificate of quality to manufacturers who meet the code requirements. The state building engineer places his name on the certificate and thus really backs up the claim that we are producing a quality product. Mr. Collins.

In our advertising we feature that we are backed by the state of Wisconsin. Because of this we have succeeded in placing our product in a much better class of buildings than we were able to before. For instance in my own business, we manufactured about 40,000 blocks of a particular type in 1923. We carried some of them over to 1924, and in 1925 we still had a few of them in stock. On the other hand in 1926 and 1927 we had one machine making this block continuously because we were getting into the class of building in which it was used. Another class of work which has opened to us is foundations for store and apartment buildings. Then the fact that we have a code with an absorption limitation that will allow cinder and other lightweight units has just opened up the lightweight market. All this progress hinges on the code.

Experience in the Use of Light Weight Aggregate in the Manufacture of Concrete Masonry Units.

By A. W. Scheer,* Milwaukee, Wis.

The Best Block Co., of Milwaukee, which has been making stone aggregate units for 13 years, became interested in the possibility of light weight aggregate for block manufacture and undertook its production, using the patented Haydite aggregate early in 1927 shortly after it came on the market in its territory. The first few months of production demonstrated the value of a light weight block and brought increasing demand for the product. This necessitated a remodeling of the plant which is now under way and a change in production methods, based on experiences and knowledge gained during the first months of operation. It is the purpose of this paper to discuss those experiences and problems raised in using this new material and to present the proposed changes in manufacturing methods.

The Aggregate.—The aggregate is manufactured by the Western Brick Co., Danville, Illinois, by burning shale similar to that used in making face brick. The shale is ground to 1½ in. particles and introduced into a rotary kiln where it is pre-heated or oxidized at a temperature that will produce a vitrified external layer on each particle. When this point is reached, the material is placed under an impinging flame which rapidly brings the temperature to approximately 2,000 deg. F. thus causing a sudden expansion of the combustion gases confined within the vitrified outer layer and causing a puffing out of the entire particle. The particles fuse to clinkers at this temperature and are discharged from the kiln as soon as possible after the expansion has taken place to prevent clogging the kiln with fused clinkers. The clinkers are crushed and screened into the graded aggregates.

The resulting product is minutely cellular, and weighs approximately 1,200 lb. per yd. The aggregate is shipped in 70-yd. cars in two grades, the fine ranging from dust to 3/16 in. and the coarse ranging from 3/16 in. to ½ in. The manufacturers supply a ¾-in. size for structural use.

Plant Layout.—The aggregate is dumped from the cars into a 12 x 12 ft. conveyor pit, where it is passed to a conveyor belt which carries it to overhead storage bins. Using this method, two men can readily unload three cars of aggregate in a 7-hr. day.

The same belt is used for conveying cement from cars to the storage floor by dropping the sacks from the car into a chute leading to the con-

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veyor belt. This has been found to be an economical handling method using the same equipment and labor for both cement and aggregate.

The belt runs on an 18 per cent incline and is entirely under cover, thus doing away with weather limitations on cement handling. The overhead storage bins empty into a 42-cu. ft. car which hangs from tracks and dumps directly into the mixer. The cement is stored on the mixer floor below the bins, thus making for an economical utilization of housing space.

Equipment and Labor.—At present the mixer operator carries the cement and dumps it into the mixer. Under the proposed method, cement will be placed on rolling platforms holding 30 sacks of cement, which will eliminate hand carrying.

The mixer is of a batch type, with 42-cu. ft. capacity and is proving entirely satisfactory. A stripper-type block machine is used. The labor requirement consists of one mixer operator, one machine operator, two carriers and one car handler.

Mixing and Molding.—Originally a 1: 1: 6 mix was used, but it has been changed to the present mix of 1: 5: 7. The change was made when more fines were added to the aggregate by its manufacturers. The first cars of aggregate received were somewhat lacking in fines, a condition which has since been corrected. Three and one-half sacks of cement are used to one mixer batch.

The amount of mixing water used varies slightly with the moisture content of the aggregate as in standard practice, but averages 35 to 40 gal. to the batch or about 10 gal. per sack of cement. Based on the water cement ratio, this would appear to be too much water, but the absorption of moisture by the aggregate seems to make this amount satisfactory and economical. The cement and aggregate are mixed dry for 4 minutes, at which time about one-half the water is added while the mixing continues; then the remaining water is added. The division of mixing water was found to be desirable for obtaining uniform distribution of moisture. The wet mixing continues for 5 min., the total mixing time being 9 min.

Block made from this light-weight aggregate sets much more rapidly than stone units. In spite of the relatively large amount of water added, the mix is not plastic, and some difficulty has been experienced in getting the block to hold together after removal from the machine. Experience seems to show that rapid setting produces a better block apparently on account of the high absorption of moisture by the aggregates, which is available later for cement hydration and internal curing.

The practice has been to tamp the block four times simultaneously. As previously stated, some difficulty has been experienced in getting the block to hold together after removal from the machine with this amount of tamping, and under the proposed method the block will be tamped seven times alternately. Experience has shown that increasing the number of tamps increases the density of the block, but also increases the weight. Since light weight is one of the important features of masonry units made from this aggregate, any increase in weight is to be avoided. Thus it is

desired to maintain a balance between lightness in weight and sufficient density for the block to stand up after being removed from the machine and before curing. The proposed increase in number of tamps from 4 to 7 will maintain a block of sufficient strength at only a slight increase in weight.

Curing.—Due to lack of curing rooms, air curing by sprinkling with water under a shed has been the method used. Present remodeling operations include the installation of steam curing rooms where the block will be cured for 48 hr.

Storage.—The blocks are piled with the cores vertical except the top layer, which is laid with the cores horizontal. It has been the aim of the manufacturer to store the block for at least 30 days, but market demands have made this difficult at times. There has been a minimum of breakage trouble with cured block.

Finished Block.—The yield of finished block has been thirty 8 x 8 x 16-in. block per sack of cement and 57 block per yd. of aggregate. Production has approximated 3,000 units per day. Blocks produced by the above method weigh approximately 25 lb., having 42½ per cent air space and exceed the Wisconsin building code specification of 700 lb. per sq. in. in strength and meet the specification for absorption.

Tests on units manufactured from this light weight aggregate for strength and durability, heat and sound insulation and fireproofing give very favorable results. The fire and fire-and-hose stream tests by the Underwriters' Laboratories resulted in a 3-hr. fire retardent classification which is the same as other classes of concrete masonry units. It is interesting to note that these tests were made under a dead load pressure of 80 lb. per sq. in., the total load on the 10 ft. test panel being approximately 39 tons. The maximum deflection toward the fire side during the tests was ⅜ in. After cooling, permanent bulging toward the unexposed side was ⅛ in. This may be compared with other concrete units which under the same test showed a maximum deflection, up to ¾ in. in some cases, although the permanent bulging after the fire and fire-and-hose-stream test was approximately the same.

Durability tests at the University of Wisconsin with 100 reversals in freezing and thawing give a 30-yr. rating. None of the block tested for compressive strength has fallen lower than the 700 lb. per sq. in. specification and range from 700 to 1,000 lb. per sq. in. As a sound insulator these units have proven superior to clay tile and to 2½-in. gypsum plaster on metal lath.

Conclusions.—The experiences of the Best Block Company in the manufacture of building block using the light-weight aggregate, Haydite, indicate that with proper equipment and proper production methods a high-grade block of satisfactory strength and durability can be produced at a reasonable cost. The product is finding a ready and steadily increasing market demand and the manufacturers consider that it has tremendous future possibilities.

Discussion.—Light Weight Aggregate.

C. A. Wiepking.—Mr. Collins in his paper explained the co-operation between the Wisconsin Industrial Commission and the manufacturers of concrete building units in Wisconsin. When the lightweight aggregates came into the market there, Mr. Muehlstein, the building engineer of the Industrial Commission, wished to have some tests made on this material so that he would have some basis upon which to rest his approval. Accordingly he asked us at the University of Wisconsin to conduct freezing and thawing tests on these materials. We made those tests, beginning with cincrete, last August. They were completed in the month of December, and when I prepared my report for the Cincrete Company on their part of the test, Mr. Pitner made arrangements for presenting some of that information here. He asked me to prepare a brief summary of the results, which I will read to you at this time. **Mr Wiepking.**

The specimens consisted of a set of 12 blocks of the 8 x 8 x 16-in. size, selected by E. B. Bushnell of the Wisconsin Concrete Products Association from stock at the company's plant. Half of the specimens were given 100 reversals of freezing and thawing; the other 6 were tested for compressive strength in the ordinary way, three at 28 days and three at the end of the freezer run.

The freezing test specimens were soaked in water, then placed in the freezing chamber until frozen solid. After freezing for a period of 5 hours or more, during which time the temperature in the freezer reduced to an average low value of 21 deg. F., the blocks were moved to a tank and thawed in water at 140 deg. F. for one hour. The blocks were cooled in water and put back into the freezer in the saturated condition, and the freezing and thawing was continued in like manner until 100 reversals had been made. Compression and absorption tests were made on the specimens from the freezer after the 100th reversal, and the results were compared with the values from the specimens that had not been frozen. Standard methods were followed in all the tests.

In Tables 1, 2 and 3 are given some of the test values, taken from the original report.

From these values it appears that the specimens did not lose strength in the repeated freezing and thawing; in fact, the average strength for the freezing test specimens was higher than for the sets not frozen. The low values for loss of weight indicate that the damage due to freezing was negligible. Careful inspection of the specimens after the freezing test gave no evidences of cracking, checking, unsoundness, or any other defect that might be expected due to freezing action.

This material proves to be durable in the freezing test in spite of its high absorptive capacity. Accordingly it is believed that the value of per cent absorption, even allowing for differences in unit weight of concrete, should not be taken as a direct measure of durability. The characteristics of the aggregates must be considered in this connection.

TABLE 1.—COMPRESSIVE STRENGTHS BEFORE AND AFTER FREEZING.

Specimen No.	Age when Tested	Maximum Load, lb.	Compressive Strength, lb. per sq. in.[2]	Average Strength, lb. per sq. in.[2]	Remarks
C-3	28 days	141,090	1115		Ordinary test on block not frozen.
C-7		134,610	1063		
C-11		115,870	912	1030	
C-1	109 days	124,060	982		Ordinary test on block not frozen. Stored dry after 28 days.
C-5		136,270	1079		
C-9		134,480	1064	1042	
C-2	109 days	123,820	978		Specimens frozen and thawed 100 times. Cured 28 days before first freezer run.
C-4		164,040	1300		
C-6		156,140	1238		
C-8		118,720	938		
C-10		128,980	1019		
C-12		161,360	1267	1123	

TABLE 2.—ABSORPTION VALUES BEFORE AND AFTER FREEZING.

Specimen No.	Value of Per Cent Absorption		Increase in Per Cent	Remarks
	at 28 days	at 109 days		
C-2	21.6	25.0	3.5	Per cent absorption computed from dry weight in each case. Test on air cured block at 28 days. Specimens given 100 reversals of freezing and thawing, and absorption test repeated at 109 days.
C-4	18.5	22.4	3.9	
C-6	19.1	20.9	1.8	
C-8	20.7	25.5	4.8	
C-10	19.3	25.2	5.9	
C-12	19.9	23.5	3.6	
Average	19.83	23.12		

TABLE 3.—LOSS OF WEIGHT IN FREEZING REVERSALS.

Specimen No.	C-2	C-4	C-6	C-8	C-10	C-12
Per cent Loss	0.96	1.21	1.54	0.79	2.45	0.93
Average Per Cent Loss	1.31 per cent					

Mr. Christensen.

EINAR CHRISTENSEN.—The discussion by Prof. C. A. Wiepking presents certain tests on the resistance of cinder concrete building units to freezing and thawing action. It occurs to me that it may be of interest to compare the results of these tests with the results of other tests made over a period of years, and I believe that such a comparison will point out certain factors which affect the resistance to frost action.

It should also be of interest to consider an important investigation made by Prof. H. Kreuger, dealing with climatic action on building materials. Professor Kreuger introduces the so-called "wetness factor" as a measure for frost resistance, and his results will be found to be in agreement with the data on cinder concrete.

The discussion leads to a criticism of our present absorption requirement and suggests the investigation of the factors that govern the permanence of building materials.

FREEZING AND THAWING TESTS ON CINDER CONCRETE.

Briefly, the tests by the University of Wisconsin on cinder blocks made by the Cincrete Products Corp., Milwaukee, brought out the following facts:

The compressive strength of blocks submitted to 100 freezing reversals was, on the average, 7.8 per cent greater than the strength of blocks cured under normal condition for the same length of time. The average loss in weight was 1.3 per cent, which may be accounted for by the dissolving action of the thawing water. The absorption of the blocks at 28 days averaged 19.83 per cent by weight, and increased to 23.12 per cent after 100 freezing reversals. There were no visible checks or cracks, no loose surface particles and no indication of spalling on the block submitted to the reversals. Comparing the fractures of frozen and normally cured blocks, no difference could be noted.

A number of freezing and thawing tests have been made on cinder block and cinder brick during the past seven years, partly to meet the requirements of building codes, and partly in connection with research on new mixtures. These tests supplement the interesting results of the Wisconsin test and furnish valuable information regarding the quality factors of cinder concrete.

Some of the tests show a remarkable increase in strength of the cinder blocks submitted to freezing and thawing. A cinder block tested by the University of Toronto in April, 1925, showed an increase of 40.7 per cent after twenty reversals, while others tested by Columbia University in Dec., 1924, showed an increase in strength of 16.25 per cent, 22.7 per cent and 32.6 per cent, respectively, all based on parallel tests on air-cured specimens.

It is not necessary to quote all the tests on record. It may be well, however, to point out that the increase in strength must be due to the curing action of the thawing water, and that the significant fact is the absence of cracking, spalling or other evidence of disintegration. In case of the abnormally high increase in strength, the specimens must have been cured at rather low temperature up to the time of the freezing test, thus showing a particularly large gain by the exposure to the thawing water.

The tests seem to justify the following two statements: (1) That strength is not synonymous for quality; and (2) That the standard absorption test gives no information regarding the resistance to frost action.

Compare for example two series of tests made by the Rochester Municipal Laboratory, July, 1923, and Columbia University, Oct., 1923. In these series cinder brick were tested, made by the same process, but differing in compressive strength and density:

	Rochester brick	Columbia brick
Freezing reversals	42	20
Compressive strength before freezing	3,290 lb.	4,920 lb. (aver.)
Absorption by weight	15.8 per cent	6.34 per cent
Absorption adjusted for weight according to A. C. I.	11.5 per cent	4.65 per cent
Change in strength by freezing and thawing	2.2 per cent INC.	24.1 per cent LOSS

If these brick were compared on the basis of strength and density, the Columbia brick would have been considered far superior and would be judged to be a concrete brick of exceptional quality. As a matter of fact, the Rochester brick would be rejected if the A. C. I. specifications were followed, as absorbing too much water. In resistant to frost action, however, the Rochester brick proved to be far more permanent, and it may be doubtful that the Columbia brick would have survived 42 reversals.

In Dec., 1921, the writer conducted certain experiments on the use of finely ground cinders in the making of smooth-face brick. The maximum size of the aggregate was less than ⅛ in. These bricks were tested by Columbia University and showed an average compressive strength of 3,035 lb., and an absorption of 13.61 per cent by weight, or 10 per cent adjusted for weight in accordance with A. C. I. specifications. In other words, the brick passed the requirements for strength and absorption, but when submitted to freezing and thawing began to disintegrate at the fifth reversal and were completely destroyed at the twentieth reversal.

Compare with this the Wisconsin block. Adjusting its absorption according to the weight, (84.5 lb. per cu. ft.), it is found to be 12 per cent, and the blocks would therefore have to be rejected by the A. C. I. specification as "not weatherproof," while the above brick of 10 per cent absorption would pass. The Wisconsin blocks, however, passed 100 freezings and showed an increase in strength of 7.8 per cent.

It seems obvious from this that the strength requirement and the absorption limit give no information regarding the resistance to frost action, at least as far as cinder concrete is concerned. Their value as a measure for other qualities will be discussed later, but we shall first take up the question of frost action.

POROSITY, ABSORPTION AND WETNESS FACTOR.

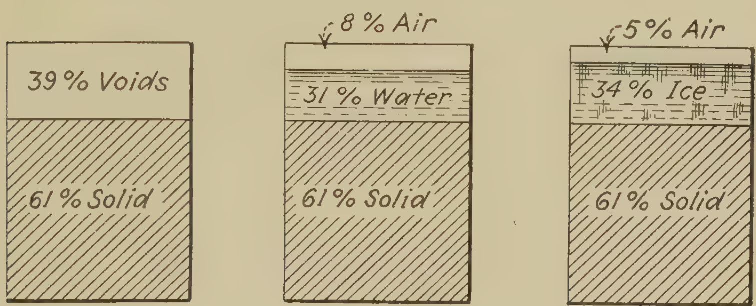
Most building materials are porous, containing a certain number of cells usually filled with air or water. The porosity may be expressed in

percentage of the total volume of the specimen—a percentage which we shall here designate by the letter P.

Even when the specimen is submerged, the pores are rarely filled completely with water. The amount of water taken up by 4 days' immersion may perhaps be considered an approximate expression for the ultimate absorption under normal conditions. We shall here express it in percentage by volume and designate it by the letter W.

If a specimen containing water in its pores is subjected to freezing, the water will expand and will, when frozen through, have increased approximately 10 per cent in volume. It would appear that if the pores and the water are distributed uniformly throughout the material, there will be room for expansion if only 90 per cent of the volume of pores were filled with water before freezing. In that case the material should be frostproof if W/P is less than 0.90.

The problem is graphically illustrated in the accompanying illustration which shows the relative percentages of solid material and voids in a fairly typical cinder concrete. It also shows the volume of water that may be absorbed by this particular concrete, as well as the volume of the ice formed from this water by complete freezing. In this case 5 per cent of the total volume are still open voids after the water is frozen.



VOIDS AND WATER AND ICE CAPACITY OF A TYPICAL CINDER CONCRETE.

The factor W/P, and its importance as a measure for the resistance to frost action, was first suggested by Professor H. Kreuger. It will be seen that, according to the above reasoning, the resistance to frost action is not dependent on the absorption alone, nor on the porosity alone, but on the ratio between the water absorbed and the total porosity of the body.

We shall here call W/P the wetness factor. W is the amount of water absorbed by 4 days' immersion of the specimen expressed in percentage by volume. Later we shall discuss whether this test is fair to materials of low capillary attraction. P is the porosity of the specimen expressed in percentage of the total volume. It may be determined as follows: First, determine the volume of the specimen by measuring or by

the displacement method. Then crush the specimen to pass No. 200 mesh and determine the specific gravity of the powder by the use of kerosene in the ordinary Le Chatelier flask. By deducting the actual volume of solid from the volume of the original specimen, the pores may be determined. As an example may be quoted a test made in Toronto on cinder concrete blocks:

	A	B
Volume of specimen	495.5 cc	715 cc
Volume of solid material	302 cc	447 cc
Volume of voids	192.5 cc	265.5 cc
Volume of water absorbed	154.0 cc	205 cc
P	38.8 per cent	37.2 per cent
W	31.0 per cent	28.8 per cent
W/P	.80	.77

INVESTIGATION BY PROFESSOR KREUGER.

This investigation deals with climatic action on the exterior of buildings, and is reported in No. 24 of the proceedings of the Swedish Academy of Engineering Science, published by A. B. Gunnar Tisells Tekniska Forlag, Stockholm, 1923.

The scope of the investigation is large, but we are here mainly concerned with the question of frost action. The writer called attention to the paper in a previous study of cinder concrete building units, published last year, and it is understood that Committee C-3 of the A. S. T. M. is now taking steps to have the paper translated.

In his investigation of the wetness factor, W/P, Professor Kreuger reaches the following conclusions:

> First: That neither the absorption nor the porosity alone may be considered an expression for the resistance to frost action.
>
> Second: That the resistance to frost action is dependent on the shape, size and distribution of the pores.
>
> Third: That a building material may be considered frostproof when the wetness factor, W/P, is less than .85.
>
> Fourth: That the material in the wall, exposed to the weather, may well contain as much water as it would take up by 4 days' immersion. At any rate, this should apply to the outside layers of the wall.

Professor Kreuger applies his wetness factor to 55 different brands of clay units submitted to 25 freezing reversals. In these instances he takes the loss in weight as a measure of the frost action, a procedure which may not be justified in case of concrete where the dissolving of part of the body does not of necessity indicate a weakening of the material, as shown by the Wisconsin tests on cinder blocks.

In the tests quoted by Professor Kreuger, the loss in weight was due partly to the spalling off of larger pieces and partly to the loosening of smaller particles of the material. It is unquestionably true that very fine cracks in the wall create planes of weakness which cause the wall to suffer particularly from frost action. Examples of this kind are very frequent.

Arranging the clay products according to the wetness factor he finds:

Group	Wetness Factor	Number of specimens	Av. weight loss
1	0— .69	8	.0 per cent
2	.70— .79	10	.4 per cent
3	.80— .84	13	1.2 per cent
4	.85—1.00	23	3.5 per cent

It will be seen that the group having an average loss in weight of 3.5 per cent is by far the greatest. The highest loss was 30.8 per cent, the second highest 17.1 per cent—which may be considered equal to practical destruction.

If the 55 samples of clay products are arranged according to the absorption of the specimens, it will be found that this arrangement gives no idea of the relative resistance to frost action. For example, a unit absorbing 41.8 per cent water by volume was far more frost resistant than a unit absorbing 29.9 per cent by volume. The former had a wetness factor of .80, the latter a wetness factor of 1.00.

COMPRESSIVE STRENGTH AND ABSORPTION.

The compressive strength of a building material is naturally of the greatest importance in determining its structural fitness. In case of masonry units, the ratio of wall strength to unit strength and the question of the stability of the wall should be considered.

Strength, however, is not synonymous for quality and, as far as concrete building units are concerned, the necessary strength is readily obtained. It is my experience that too much emphasis is laid on the structural strength of a masonry unit, with the result that we forget the other functions which the unit has to perform in the wall.

Briefly, the perfect wall must be permanent, it must possess a certain amount of heat insulation and sound absorption, and it must not be prohibitive in cost.

Placing before an engineer the two products previously discussed as the "Rochester brick" and the "Columbia brick" he will generally select the latter as being the best unit. Its strength is much greater, its density is much greater, therefore it has a greater factor of safety. But the factor of safety in a 5,000-lb. brick is uncalled for in ordinary construction, and the 3,000-lb. brick will perform just as well. Furthermore, the latter brick is cheaper to make, is more insulating, lighter in weight, and far more permanent when exposed to the weather.

I have no criticism to offer regarding the 1,500-lb. requirement for concrete brick or the 700-lb. requirement for concrete block and tile. Past experiences seem to have justified these limits. But I am daily concerned with the erroneous idea that any strength in excess of these requirements is an indicator of superior quality.

The absorption limit of the A. C. I. specifications is a more serious matter. It has been pointed out in the foregoing that the absorption does not furnish any measure for the weather resistance of a product, and the investigation by Professor Kreuger supports this contention generally. An absorption limit which would brand as "not weatherproof" a building unit successfully passing 100 freezing reversals is obviously absurd. This seems to have been realized by Committee C-3 of the A. S. T. M. in omitting the absorption limit from the tentative standards proposed at the 1927 convention. It is to be hoped that the American Concrete Institute will follow with a similar action, or at least submit the problem to a serious investigation. The present limit is arbitrary and evidently wrong in view of the facts on hand.

It is probable that there is a certain relationship between strength and density in case of concrete made from a dense aggregate. No such relationship has been found to exist in concretes made from cellular aggregate like cinders. If it did exist, the strength requirement alone would be sufficient.

SUGGESTIONS FOR FUTURE RESEARCH.

Professor Kreuger's investigation of the permanence of building materials involves a tabulation of weather conditions throughout Sweden, an examination of existing buildings and a laboratory study of the factors which are found to influence the resistance to climatic action.

It is not within the scope of this discussion to consider other questions than those related to frost action. At the same time, I cannot refrain from pointing out the need of a general study of climatic action on building materials in this country. Natural stone, clay products, concretes made with dense and cellular aggregate, stucco and other exterior finishes are used throughout the United States, and evidently supposed to be equally permanent in all places.

With particular reference to the wetness factor, W/P, the following questions need study:

(1) Amount of water actually taken up by a building material when exposed to the weather. The distribution of this water when the wall has reached an equilibrium. The possible movement of this water due to temperature differences and other factors.

(2) A laboratory test to determine the true absorption W. In the foregoing we have assumed that W is equal to the amount of water taken up by a 4-days' immersion test, and this seems to be true for materials like clay products of high capillary attraction. It is doubtful, however, that it applies to cellular materials. Cinder concrete, for example, has a

limited capillary attraction due to the shape of its pores. Cinder blocks have been placed in water to a depth of 1 in. and left in this condition for a month or more without raising the water to a height greater than 20-25 per cent of the specimen. In the same manner, a steady stream of water has been applied to exterior of a cinder block wall for days without penetrating the wall. The action of water under pressure, or water subject to the force of gravity, should not be confused with the action of water subject to capillary force. At the same time, as long as the exterior layers of a wall may be saturated, frost action may take place. For that reason it may be found that the 4-day immersion test is justified.

(3) A further study of the porosity of building materials in case of concretes made from cellular aggregate. It should be investigated whether all of the pores are open to give room for the expansion of the water by freezing.

(4) Parallel tests on resistance to freezing action, porosity and water absorption, to study the value of the wetness factor, W/P, as a measure for permanence. It would be of the greatest interest to conduct freezing tests on small wall sections, moistened under conditions resembling natural exposure to rain and temperature changes in the proximity of the freezing point. Microscopic investigations should be made in order to determine the influence of size, shape and distribution of the pores.

I. OESTERBLOM.—A conference about light-weight aggregates would not **Mr. Oesterblom** be complete, unless the lightest of all were mentioned, namely, the common air around us. The use of air for an aggregate is specially important, because it creates a very different concrete with properties distinctly different and a combination of properties not often found in one material. Take "Aerocrete" for an example, which is an aerated or expanded concrete. The use of this material solves one of the old and ugly problems of engineering in a most graceful manner. It has light weight (standardized at 50 lb. per cu. ft.); it is fireproof and very specially so, more fireproof than ordinary concrete; it is heat resisting; it has distinct structural value; and it is at the same time very specially sound proof. The above combination of properties is unusual and offers a solution of the old problem of how buildings should be properly sound-proofed without the sacrifice of strength and fireproofing ability.

At Massachusetts Institute of Technology experiments were made showing that there was only three-tenths of one per cent sound transmission through a 3 in. plastered Aerocrete partition. This means, translated into common terms, that the most infernal noise that could be produced on one side of a partition would be barely perceptible on the other side. It also means that the structural frame could be completely isolated so that all absorbed vibrations would be transmitted into heat energy and slowly dissipated.

There are many other materials which would stop the sound as well, but none which at the same time are fireproof and have a structural value. The strength of Aerocrete reaches as high as from 400 to 600 lb. per sq. in.

in direct compression and even more in bending. This makes it more than satisfactory for partition work. Three-inch partitions will stand up well as high as 13 ft. in height, and are far better heat and sound insulators than 4 or 5-in. partitions of any other structural material now in common use. Fireproofing tests have been made at Columbia University showing only a small rise of temperature above normal with a 1700 deg. fire sustained for 4 hr. on the far side of a 4-in. Aerocrete slab. The fire stream tests were also satisfactory and the erosion small. Incidentally, the slabs were reinforced the same as ordinary concrete and were loaded with 600 lb. per sq. ft. as a matter of special curiosity. The spans were about 8 ft. center to center, and there were no signs of distress at any time.

Heat insulation is also excellent. As compared to ordinary concrete it is from 8 to 9 times as effective and 4 times as effective as brick. It is even better than wood or gypsum, and most excellent, therefore, even for outside walls. Six-inch Aerocrete walls are much warmer than 13-in. brick walls and have all the strength required for wall-bearing construction in moderate dwellings and similar structures.

Aerocrete is as waterproof as ordinary concrete and will absorb water only to the extent of 1 in. or so, if used as outside walls. Aerocrete offers an unusually good bonding surface for both cement stucco or common wall plasters. The material can be pre-cast or field-cast to suit convenience. It can be sawed into blocks by an ordinary carpenter's cross-cut saw, it can be split by a trowel, or chipped, and formed to any shape. Holes can be drilled as into wood and nails also driven with a fair holding power.

Mr. Wilk. J. C. Wilk.—I would like to ask how the selling price of these blocks compares with that of ordinary blocks?

Mr. Wilk. Benjamin Wilk.—I would like to ask about the segregation of the aggregate, both at the time the car is unloaded into the bins and at the time the car is taken out of the bins?

Mr. Franklin. Jack Franklin.—In reply to that first question, I will say that the price on 8 in. units varies from 15 to 16c. for a gravel block; Haydite is selling for 18c. In reply to the second question, there was at first some trouble in the grading of the aggregate. Then trouble with segregation was experienced when elevating the aggregate and dropping it into the bins. The discharge point is now placed as low as possible to minimize the drop, and segregation does not now occur.

Mr. Wilk. Benjamin Wilk.—I would like to ask Mr. Christensen how they determine the porocity.

Mr. Christensen. Einer Christensen.—They determine that by first determining the volume of the specimen. Then, after grinding to pass the 200 mesh sieve, they determine the specific gravity which makes available the volume of the actual solid material in the specimen. This deducted from the volume of the specimen gives the void space.

Mr. Harrington. W. C. Harrington.—I would like to ask Professor Wiepking if he has ever had any similar experience on other aggregates in cinder concrete so far as freezing and thawing tests are concerned?

C. A. WIEPKING.—The only two that were used in this test were haydite and cinder concrete, and the results of the tests showed that both of these materials passed the freezing test at 100 reversals. The Industrial Commission has approved these materials for regular use under the Wisconsin building code on the basis of these tests. I have had no experience with any other materials than those. Mr. Wiepking.

J. C. PEARSON.—I think this is a most interesting discussion that we have had from Mr. Christensen, because it points out some of the fallacies in this absorption test. I am struck by the analogy between this absorption test and the commonly specified tensile test on portland cement. They are neither of them any good, but they are used because they are easy and cheap to make. The absorption test is really worse than the tensile test, because the use of the many types of waterproofing materials may upset the results entirely. Mr. Pearson.

S. H. INGBERG.—In determining the porosity of a product for the purpose of obtaining the wetness ratio, it appears doubtful whether the whole void space inside of the unit should be included. I believe the work of Professor Kruger was based mainly on clay products where the void spaces were quite small, sufficiently so to draw water by capillary attraction. In the case of these larger void spaces, it does not appear that the same conditions apply; they are too large for capillary action and they simply are spaces outside of the structure of the body as such. Another point relating to porous products refers to what might be called storm resistance as distinct from resistance to disintegration from freezing and thawing. A product may be satisfactory from one standpoint and still be too porous for use in a driving rain. Mr. Ingberg.

EINAR CHRISTENSEN.—Professor Kruger's investigations dealt mainly but not altogether with clay units. In regard to cinder concrete, I find that while many of the cells are more or less closed, you can nevertheless force water into them. I think that is what explains the increase in absorption which Mr. Wiepking reported from the Wisconsin tests. Evidently some of these pores had been opened up. It is an important matter to study the distribution and shape and nature of the pores, to find out whether this wetness factor is of any significance. Mr. Christensen.

C. A. WIEPKING.—We are apparently getting down to the point where we are studying the internal structure of concrete and concrete units, and whether we have good results right now or not does not matter so much. The encouraging point is that we are beginning that kind of a study. Just as the study of the internal structure of iron and steel has caused tremendous progress in that field, just so will this study of the internal structure of concrete result in great advances. Mr. Wiepking.

I. OESTERBLOM.—Especially in reference to this internal structure, I would like to refer to some of my experiences which have been to a very large extent in the tropics. In India, I found, much to my surprise, that the very best engineers refused to become enthused about concrete. They said that concrete was a most unreliable product, and particularly so when Mr. Oesterblom

they had to deal with water construction. I found that the destruction was particularly severe on the western coast of India and in the southern part of Ceylon. I also found some severe destruction in Mysore and Singapore, but I found that it was not so bad in Calcutta and that fairly good results were obtained in Burma. I believe—and I mention it because I think it has some reference to this internal structure—the trouble was that during the 3 or 4 months of monsoon season water soaked into all the concrete. After that comes the sun to burn and dry everything out. This continues for several months. The change from wetness to drought destroys the concrete. This explains, I believe, why the engineers over there refuse to have anything to do with concrete. If this association is taking up the study of internal structures I thought possibly these few words in reference to this matter might be of interest, and that they might be kept in mind in carrying on the study.

Drying Concrete Brick to Take Out the Shrinkage.

By L. E. Grube.*

With the increasing necessity of late fall and early spring productions, we were confronted with a peculiar situation in the manufacture of concrete brick, that is, the drying of the brick after the steam cure process. Under ordinary summer weather, we had experienced no difficulty, but after the cold weather set in, it at once became difficult to dry the brick, so as to make ready for delivery or for the stock pile purposes. This was particularly true when outside weather conditions dropped to about 50 deg. F. or below. The first thing that came to our minds was the fact that for concrete brick or concrete units to dry, it was necessary to have air currents. It was also remembered that summer temperatures were on an average of about 75 deg. F. Accordingly, besides an electric-driven fan, capable of changing the air in the curing tunnel about once every forty-five seconds, provision was made for heat by installing steam coils.

The length of each of the four curing tunnels is approximately 55 ft., the width about 8 ft., the height on one side about 6 ft. 9 in., and on the other side about 6 ft. This gives the roof of the tunnel a slope of about 9 in., providing drainage for the condensed steam.

The fan was mounted on a rack about 5 ft. from the floor. The rest of the rack was encased with a 15-in. high opening at the bottom, so that the fan would get air from the floor, take it through the encased rack and blow it into the tunnel. This arrangement did not prove satisfactory, and a unit-heater was substituted.

A unit-heater resembles an automobile radiator. It has copper tubes that run in a horizontal direction and around each copper tube is wound a helical brass fin, which increases the heat radiating surface about 300 per cent. The unit-heater that we have installed is about 30 in. sq., and about 10 in. thick. Steam is admitted at the top, and the condensation drained at the bottom, through a trap, which returns the condensation back to the boiler. This heater as installed, can deliver 500 sq. ft. of radiation. According to the size of our tunnels, the heat required to bring the tunnel to 75 deg. F., with zero temperature outside, is 125 sq. ft., which takes into account the evaporation which will take place in order to rid the brick of the moisture retained from the steam cure. Thus we are getting four times the amount of heat necessary for the size of the tunnel.

The heater was mounted on a portable chassis and directly in back of it on the same chassis, the fan, which we had used previously, was placed. This heater and fan combination was placed as near to the floor as possible

*Sheboygan, Wis.

and still permit the condensation from the heater to drain back to the boiler. The air supply was to be taken from the outside, so we encased the fan air tight, and then cut a hole 12 in. in diam. in the rear of the box. We also cut a 12-in. hole in the tunnel door and put a 12-in. galvanized iron sleeve from the box through the door, to give us our source of fresh air. On the other end of the curing tunnel we built a 16-in. sq. air duct, from which the escaping air was led into the shop.

For those times when it was necessary for production purposes to force the entire drying system we also arranged an exhaust fan that could be attached to the air duct by means of a galvanized iron sleeve. This system working satisfactorily, we made the whole system portable by putting casters on the heater and fan combination as well as on the exhaust fan. From this it can be seen that all that was necessary was to pipe the steam lines to every tunnel, and then just move our portable drying system. Having tested this system for about a year, we find that we are getting drier bricks with this system than we do with ideal summer weather.

As yet I have said nothing about the shrinkage. It may seem, when you look at a concrete brick on a stock pile or on a job that it is dry, but most generally, like other beauty, it is only skin deep. The surface to a depth of about ½ in. is dry, but from that point to the center of the brick you will still find moisture. If bricks such as that are laid in a wall, and weather conditions are dry, the thing that results is that the brick dries out, and as they do, they shrink, resulting in cracks in the wall. The cracks in the wall are not always along the mortar joint, but most generally through the unit itself. Many cracks in concrete masonry walls are due to this very cause. However, with the drying scheme just outlined we have found through close observation that bricks so dried show very little moisture in them, a small area in the center of the brick about the size of a quarter being all that can be found at the most.

In this discussion I have referred only to concrete brick, since we are manufacturing only brick. Nevertheless, the same scheme can be used on any concrete masonry unit.

Discussion—Concrete Brick

Benjamin Wilk—Are these brick built on one pallette? Mr. Wilk.

L. E. Grube.—One pallette. Mr. Grube.

Benjamin Wilk.—Do you find a difference in the moisture content at the points where the brick is in contact with the pallette from those where it is free from the pallette? Mr. Wilk.

L. E. Grube.—No sir. Mr. Grube.

Newton D. Benson.—Is this heating process necessary in warm weather, or is it only applied through the winter? I have been making brick for some time and I find that by letting the brick cure 40 or 50 days in the stock pile in good weather I experience no difficulty of that kind. Mr. Benson.

L. E. Grube.—We have used the drying process chiefly during cold weather, although when we have peak conditions we use it in the summer as well. Mr. Grube.

Newton D. Benson.—How long do you store the brick in the yard after your drying is finished? Mr. Benson.

L. E. Grube.—Our storage with this system is probably at a minimum. Often we take the units from the drying process direct to the job. Our stock pile at no time contains more than 200,000 bricks. Mr. Grube.

Newton D. Benson.—Is your outside stock pile under cover? Mr. Benson.

L. E. Grube.—Yes, under a shed. Mr. Grube.

C. F. Chapman.—I did not notice in the paper, but I understand that there are four curing rooms and that in drying they move the drying system from one to another. How long do you leave it in each one? Mr. Chapman.

L. E. Grube.—With the size of our curing tunnels, it takes about 24 hours to dry. Mr. Grube.

J. L. Miner.—Has this process been tried on large-sized units, and if so, has it been found necessary to vary the time of curing? Mr. Miner.

L. E. Grube.—I have only tried the system on bricks. However, I believe that the scheme is being used for units as large as 12 in. blocks. Mr. Grube.

Heavy Duty Concrete Floors.

By C. E. Covell.*

The subject discussed in the ensuing paragraphs represents one of the most troublesome of the many minor problems which fall to the lot of building contractors. Briefly stated this problem involves the construction of concrete floor surfaces which will not dust or show an undue amount of wear over a reasonable term of years under the severe and sometimes exceedingly destructive service incident to modern industrial occupancy.

A good concrete floor for ordinary industrial use must embody two essential characteristics: First, strength to resist compression and shear in the slab due to static loads of machinery and stored materials and to the concentrated wheel loads of moving trucks; and, second, resistance to surface wear and dusting due to impact and the abrasion and crushing of the surface particles caused by truck wheels, the shoes of the workmen and loads dragged over the surface.

The first of these requirements is easily attained by using a proper slab thickness, adequate reinforcement and care in producing a uniform concrete of predetermined strength. The second, however, is far more difficult to secure even when extreme care is exercised. This is especially true in industrial building where the processes require the use of steel wheel trucks which not only tend to crush the surface but also to cut and gouge as the trucks are turned.

It may be conceded that under ideal conditions of temperature and humidity and where exceptionally good aggregates are readily available, special processes of surface finish and even standard methods can result in concrete floor surfacings which will compare favorably with any known substance ordinarily used as flooring materials. The real problem, however, is to produce invariably good results with materials found within easy hauling distance of the job and at a price little, if any greater, than the cost of the usual methods of finishing. In spite of the fact that floors are the only part of a structure which take heavy and continuous wear and that they are difficult to repair without interruption of the operations of the plant, most owners hesitate to spend extra money for finishing processes which will provide additional durability.

Concrete is a singularly attractive medium for finished floor construction because of its inherent possibilities of economy in first and ultimate

* Eastern Construction Manager, The Austin Co., Philadelphia.

cost, fire-safeness, high resistance to all ordinary industrial processes and a degree of sanitation not possible in more absorbent substances. It is the only flooring material which can simultaneously combine the functions of a wearing surface and a load-carrying integral part of the structure itself. Its use in increasing millions of square feet annually of floors attests its present popularity. Yet all of these advantages, and even its continued use may be jeopardized unless some economical methods can be devised to overcome its one fault, namely, this all too frequent tendency to dusting which often starts soon after the floors are placed in service and which indicates a more or less serious lack of resistance to wear.

In a construction experience which extends back over a term of more than fifty years, the company which the author of this paper represents has laid millions of square feet of concrete floor surfacings in buildings of industrial type. At first these floors were built of foreign-made cements but within the past twenty-five years the advent of high quality American-made portland cement has resulted in its substitution for the foreign-made product. Observation indicates, however, that the origin of the cement does not play a material part in the problem.

Naturally an experience of fifty years' duration has resulted in a certain amount of improvement in the initial processes and methods of building floor surfacings but it is a curious fact that in spite of all that has been observed and learned as to the behavior of concrete floor surfacings the early problems of dusting and excessive localized wear are as fresh today as ever. Even under the best recommended practice of building this type of surface, the degree of uncertainty is still very great. In fact, our experience over the past five years seems to indicate that it is virtually impossible to obtain a surface of uniform freedom from dusting and resistance to wear under the usual practices of laying whether the surface be a monolithic finish or one using a separate top course of similar composition. Our experience in this regard seems to be similar to that of others with whom we have discussed this problem.

In 1923 we undertook a rather extensive survey of the whole question of concrete floor surfacings with the result that the following facts were developed:

1. The cost of concrete floors in our standard buildings averaged 15 per cent of the total cost of the structures.

2. Placing these floors in the ordinary accepted fashion resulted in a considerable expense annually for repairs and replacements.

3. Floor repairs and replacements could offset the profits on any job and in some cases exceed them.

Such a condition certainly demanded immediate and definite action. As a first step, we got in touch with the Portland Cement Association and outlined the situation. We received their whole-hearted co-operation. After considerable study with them, we published a bulletin on concrete floors for the use of our superintendents which reviewed the results of our past experience and gave definite instructions on the methods of handling

certain features of concrete floor work. The features which received special stress were:

1. That no concrete should be poured using a wet and sloppy mix but that it should have such a consistency that screeding and floating would be required to bring water to the surface.

2. That concrete must be mixed for not less than 1¼ min. and longer,

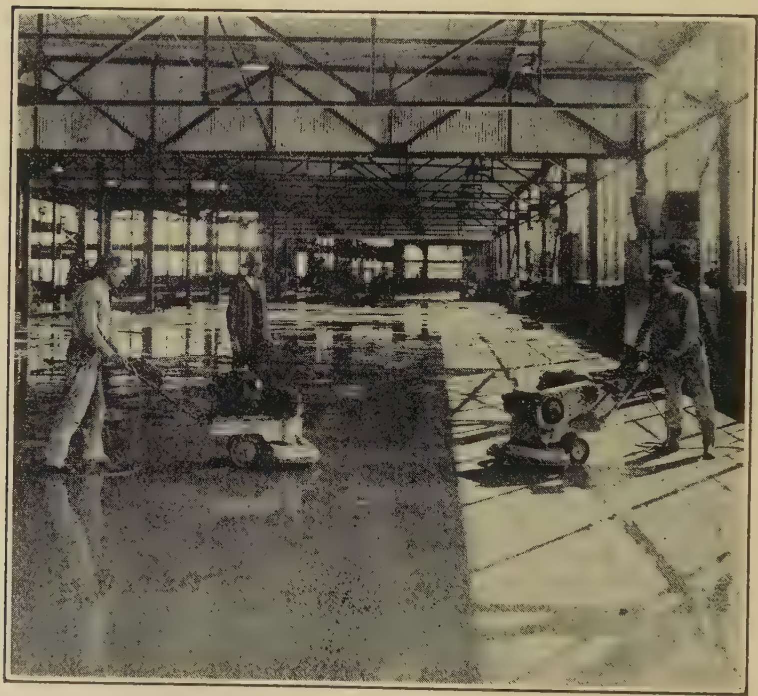


FIG. 1.—GRINDING FLOOR OF AN INDUSTRIAL PLANT.

Bottom casting turns on vertical shaft. Carborundum stones are used as the grinding material. Floor is kept flooded with water to facilitate grinding.

if possible, up to 2 min., calling attention to the fact that concrete mixed for 2 min. was about 35 per cent stronger than when mixed for ½ min.

3. That properly graded aggregates was very necessary and that aggregates for floor work must be strictly in accordance with our specifications.

4. That proportions specified must be followed strictly and that no attempt be made to save cement.

5. That it was very important to test the fine aggregate for organic impurities in accordance with instructions given.

6. That only trained, qualified men were to do the floor finishing.

7. That floors were to be troweled as little as possible commensurate with a smooth surface.

8. That absolutely no "drier" was to be used in surface finishing.

9. That floors should be properly cured for a period of at least 10 days, or longer, if possible. Data of the Portland Cement Association were furnished and attention directed to the fact that proper curing increases the strength and resistance to wear very materially.

FIG. 2.—PLACING THE 1-IN. TOP COAT OF 1:2 MIX IMMEDIATELY AFTER THE POURING OF THE BASE.

Surface is wood-floated and then steel-troweled to prepare for grinding.

We soon registered a marked improvement in the quality of our work, but even after this determined effort to secure better floors, we found we had to go still further to get the desired results if we were to continue to use concrete floors under rather severe trucking conditions.

With this in view, we made tests on many different sands available for concrete work all over our eastern district and found in every case enough very fine soluble or insoluble matter to authorize the statement that there is no commercial concrete sand available in the East that we would say is 100 per cent suitable for the wearing surface of concrete

floors which are to be monolithically finished by screeding, wood float, and, finally, with a steel trowel.

In examining these sands, we used test boxes 2 ft. square and 4 in. or 5 in. in depth which were filled, screeded, floated, and troweled in the usual floor-finishing manner. In almost every case, the very fine matter

FIG. 3.—MACHINE USED IN MAKING ABRASION TEST ON FLOOR AT NORRISTOWN, PA.

Note the weight over each wheel.

contained in the sand was brought to the top or wearing surface of the slab by this ordinary process of finishing. On several test blocks ⅛ in. of very fine material was deposited in the bottom of the box before the coarse aggregate was put in, this fine material being removed from a larger amount of the coarse aggregate by screening. It was found that the

suction developed by the floating and steel trowel finishing was sufficient to bring a large percentage of the fine material up to the top of the slab, especially if an excessive amount of water was used in mixing.

Further (even when this excess amount of water was very slight), we found that it had the effect when brought to the top by screening and floating of carrying off the cement in the very top of the slab after the darbying process, leaving the wearing surface virtually devoid of cement. Then, when the cement drier was applied in accordance with the usual

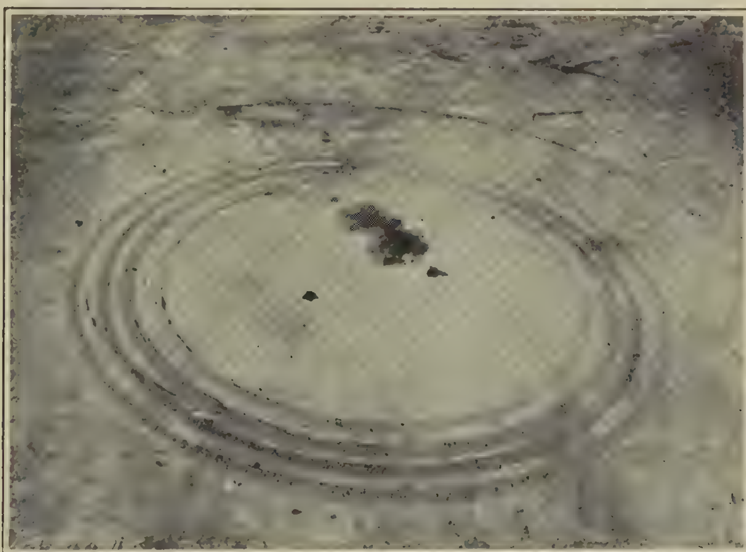


FIG. 4.—FULL VIEW OF TRACKS MADE BY WHEELS IN ABRASION TEST.
Note small amount of wear and the sharp edges of tracks.

practice, to take care of the remaining water on top of the slab, or to get the slab in a workable condition, there was formed what appeared to be a very hard surface under the steel trowel but which in reality was composed of only the drier on top and directly beneath the fine material which, as before described, had been brought to the top of the slab. These tests but confirmed our observation of the causes of failure of some actual floors.

These tests also proved that under no circumstance can a drier be safely used on a concrete floor later than one-half hour after the concrete has been deposited in place. We all know that it is a common practice of cement finishers to use a cement drier sprinkled over the top of the slab to absorb excess water as late as one to three hours after the mixing and

placing of the concrete. Our tests proved conclusively that it is absolutely impossible to get a hard wearing surface when this practice is followed.

With the idea in mind that it was entirely reasonable and possible to improve the quality of concrete industrial floors at no increase in cost and having made these tests and observations, we came to the only natural conclusion, i. e., that it is simply a choice of two methods:

1. To secure a sand that is absolutely free of loam organic impurities, or the fine material. Our investigation of sands showed that it is possible

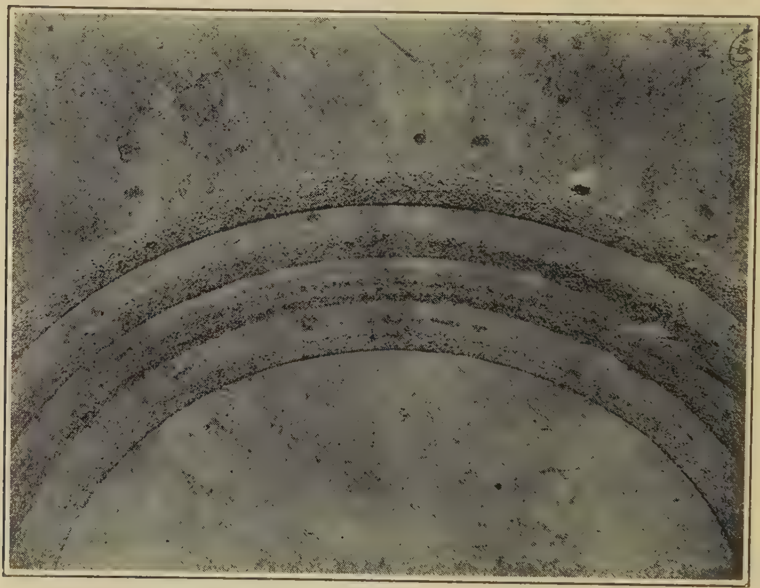


FIG. 5.—CLOSE VIEW OF TRACKS MADE IN ABRASION TEST.

Texture of worn surface is exaggerated. If whole floor were to wear this way it would still be considered a good, smooth factory floor.

to get good material at three or four different points in the eastern district which would serve our entire territory in the East but that the cost would be prohibitive as there would be required the installation of special washing machinery and screens to take out the fine material and that the sand would have to go through a triple washing operation in order to be sure that all the fine material was removed. This increased cost of the sand would add from three to five cents a square foot to the cost of the finished floor depending on the location of the work.

2. To use a sand which meets the specifications for concrete sand for other types of work, if, after the floor was finished, we could remove the skin of poor material on the surface by grinding. We decided to give this second method a trial and first used it on an industrial plant in Virginia.

In this case we had a monolithic floor laid directly on grade. The floor was 5 in. thick and unreinforced except at machinery pits. The concrete mix was 1 : 2 : 3½ using local sand which contained a large percentage of material too fine for the purpose and a crushed limestone ranging in size from ¼ in. to 1½ in. This stone contained small particles of shale or clay material. No attempt was made to place any type of topping. The concrete merely being mixed quite dry, placed in position and struck off with a straight edge, a tamper, and a long-handled darby to a fairly level surface. The floor was cured under wet sawdust. The grinding was started when the floor had cured for 5 days.

The finishing process was carried on by the use of three hand-operated electrical grinding machines and sand as an abrasive material was used under the cutting heads of carborundum to assist in producing a smooth surface. Plenty of water, however, was used during the grinding operation. No attempt was made to produce a true and even-textured surface, the desire being merely to remove all dirt, laitance, and float marks left by the straight edge and darby. This resulted in a floor which showed areas resembling a fairly good trowel job and other areas where the aggregate was exposed. There was a small amount of pitting due, in all probability, to the clay content of the aggregate and possibly to accumulation of fine sand in pockets which were picked up under the grinding.

Accurate costs were kept on the work and it was found that this method of finishing was costing only slightly more than one cent a square foot, including labor, machinery rental, supplies, and power. The cost of the rough base was also quite low because little if any overtime was required in producing a surface smooth enough for grinding. From this first job and several succeeding ones where the floors were laid in about the same manner, and totaling about 1,000,000 sq. ft., we learned a number of important facts, viz.:

1. At least ten days must be allowed to *water* cure the floor before grinding it.

2. Coarse aggregate containing any considerable amount of flat elongated particles or of soft particles will cause pitting of the surface when the floor is ground.

3. The top surface of the concrete floor must be of a richer mix than that obtained by the use of a 1 : 2 : 3½ or 1 : 2 : 4 concrete.

4. The use of a smaller maximum size of aggregate in the top will give the finished floor a more uniform appearance.

We then revised our specifications to incorporate these changes and the floor of the building for the Wildman Manufacturing Company at Norristown, Pa., was laid with a base course 4 in. thick of 1 : 3 : 5 concrete, placed with 2 in. slump. This base was leveled off and immediately a 1 : 1 : 1 top coat, consisting of cement, concrete sand, and pea gravel was applied to a thickness of 1 in. This top coat was placed also with a 2 in. slump. This topping was leveled off with straight edge and finished with a wood float and a single application of a steel trowel. The surface was

then covered with wood shavings and kept wet for 10 days. At the end of this curing period, grinding was started, it being done from 10 days to 3 weeks after the placing of the concrete. This floor turned out to be in appearance and apparent hardness, vastly superior to any floor we have ever laid, the surface being smooth, hard, close-grained with apparently no dusting. Average costs for all of the floors finished in this manner to date approximately 1,200,000 sq. ft. are under 3½ cents per sq. ft. which includes cement finishers' labor for straightening, floating and finishing and all expense of grinding, viz.: grinding machine rental power, grinding discs, etc.

Floors using a separate 1:2 topcoat were placed so recently that naturally we did not have any information as to its resistance to actual severe trucking and other abuses of factory operations over a period of years, but to obtain an idea of its wearing quality a trucking or abrasion test was conducted about a month and a half after the floor was completed. The apparatus consisted of a heavy table about 30 in. high supporting a motor and reducing pullies and shafting to give a slow turning motion to a vertical shaft placed near the end of a heavy timber extending from one corner of the table. This shaft near its upper end ran in a box fastened to the side of the timber and its lower end was held in place in a foot-block fastened to the floor. Near the bottom of the shaft was keyed a horizontal plate from which extended, 90 deg. apart, four arms fastened loosely to the plate so that they were free to move up and down slightly. Underneath the outer end of each arm was fastened a steel truck wheel. These wheels were 2¼ in. wide and 6 in. in diameter and the wearing face was slightly convex. The wheels were spaced so that in turning they described circles of 48, 44, 40, and 36 in. in diameter. Therefore, no two ran in the same groove. On the arm over each wheel was placed a weight of 100 lb. This would equal a live-load of 300 lb. on the common three-wheel factory truck wheel which is probably more than the average actual load.

The apparatus was geared to give the wheels a speed of 15 r.p.m. The test was continued for four consecutive days and ran for a total of 26 hours. The first appearance of dust was noticed 3 hours after the test started and gradually accumulated as the test continued. Upon the completion of the test four distinct grooves were worn in the floor varying in depth from 1/64 to 1/32 in. below the original surface. While the surface of these grooves was generally somewhat rough there was no apparent tearing up of the concrete. The wear was fairly gradual and regular over the whole surface.

What period of time this test would represent under actual conditions is, of course, indeterminate, there being such a wide variation in trucking conditions in different factories and industries. But let us assume that under the average actual condition a truck wheel would run over a given spot once an hour. Then, neglecting the constant turning action of the wheels in the test, one hour of the test would represent 900 hours of

actual service. The period of the test would represent 26 x 900 or 23,400 hours of service. Using 55 hours as the factory week, it would be equivalent to about 8¼ years of service. It is believed that this imposed a much more severe wear to the floor than would be experienced under actual conditions for most of actual trucking is done in fairly straight lines with only occasional turning while in the test the turning and gouging action of the wheels was constantly present.

In any devised test of short duration it would probably be difficult if not impossible to stimulate actual conditions and the test described we purposely made severe in order to get in a short time some idea of what the floor would stand. We feel that the floor tested is as good as it looks and that it will withstand severe trucking for a great many years without appreciable wear and that the dusting caused by wear will be so slight as to cause no concern. In the Eastern district we have placed and ground about 400,000 sq. ft. of floor similar to that at Norristown and each unit apparently is as good as the one tested.

Discussion.—Heavy Duty Floors.

Mr. DeSpirt.

A. C. DeSpirt* *(By Letter)*.—We find from our own experience and in studying the experiences of others that the grinding of concrete floors is perhaps the only solution known today which gives the desired effect of eliminating the soft and dusty surfaces of cement finish.

The process of rubbing is explained quite satisfactorily in this article, but in addition to the grinding we wish to add the following recommendation: After grinding, thoroughly scrub with iron brush and dampen the concrete floor; then spread over the floor a liquid coating of water and cement. Rub the coating into the pores of the floor with a grinding machine until it is thoroughly absorbed and set. If any cement remains on the surface, this surplus can be removed with a trowel. Spread heavily a coating of dampened sawdust over this surface. Keep same well dampened for 2 or 3 days in order that the cement grout may not be subjected to the surface air and dry too readily without having sufficient time to exercise its full setting polish.

Cement flooring is not part of our line, but we have occasionally prepared floors in this way for contractor friends at times when they experienced difficulties, and have obtained very satisfactory results.

Mr. Bird.

M. F. Bird.—I agree with the author that most owners hesitate to spend any extra money to secure in concrete a hard, dustless, long-wearing floor surface. Ordinary concrete, rodded, straight-edged, floated and trowelled will not do in the greater number of industrial structures. I believe that what is known as the metallic monolithic floor, that is, a reinforced-concrete slab with a dust coat of metallic hardener, is more resistant to abrasion than any other standard product. It is offered for sale the country over (commonly known as "iron filings") by at least four reputable companies. Comparative tests are usually difficult to obtain, due to unconscious prejudice on the part of those conducting them, but a contractor whose sole interest is to obtain the best possible results, has agreed to conduct a test on a building now under construction. This will permit a comparison of the method described by Mr. Covell with a typical metallic monolithic finish, under conditions which should give reliable data on both relative cost and wearing qualities of these two types of floor finish. If the committee on Floor Finish is interested in following the results of these tests, the information will be submitted.

Mr. Rockwood.

E. F. Rockwood.—The firm with which I am associated has, for the past 10 years, been trying to perfect floors free from wearing and dusting.

*The Italian Mosaic and Marble Co., Inc., Buffalo, N. Y.

I do not believe there is anything that is a sure cure for a bad floor, but I know that by grading these floors with a combination of gums and oils, similar to a varnish but more elastic and not so brittle, we have been able to stop almost every case of dusting.

W. E. HART.—I think there are in the records of the Institute studies on shrinkage that can be applied to floor construction with considerable advantage. It has been found that by the use of proper sized coarse aggregates in floors, it is possible to reduce shrinkage. It is my understanding that the Floor Committee is contemplating a revision of the present floor specifications. Right now they call for a 1: 1: 1 top, and I believe the experience in the last 5 years indicates that possibly a 1: 1: 2 top will reduce the shrinkage considerably. This is an important thing and one that is coming under my personal observation at the present time. In connection with Mr. Covell's paper, I would like to ask Mr. Archibald (who presented Mr. Covell's paper) if he could give us any idea as to the relative cost between trowelling and grinding a surface to a finish? **Mr. Hart.**

MR. ARCHIBALD.—Our cost of finishing the surface, including the floating with one steel trowel finish, is about 3½¢ per sq. ft. **Mr. Archibald.**

A. S. DOUGLASS.—I have recently been through an experience with metallic surfacing which I think may be recorded with value to others. The chief fault we have experienced has been occasioned by a failure of one of the outstanding metallic surface concerns to avoid an error which I believe is commonly recognized in floor laying. Their recommendation was approximately: 1 volume of metallic material to 3 volumes of dry cement dusted on at the time when the excess water had completely disappeared in trowelling. That practice of dusting dry sand on cement or dry cement has been found to be bad by many of us who have tried it. We have found exactly the same symptoms in this floor that I would expect to find where we had dusted a wet floor with dry sand and cement. The abrasion has occurred immediately in the path of construction traffic after the floor had set but before the building was completed. **Mr. Douglass.**

R. B. GOGE.—I do not agree altogether with some of the statements made in regard to the scaling or peeling of concrete. I think one of the chief troubles is with density of the top layer of concrete. The part that peels off is invariably porous, especially where a little excess water was used. To date we have had very little peeling with concrete where the thickness of the mortar surface was over ⅜ in. In those cases in which thick surfaces scale the water evidently that did appear was not sufficient to reduce the density of the concrete mortar surface. In all cases where there is a very thin layer, it would appear that the density is reduced and the thickness of that thin scale is not sufficient to resist the impact of traffic. If the original concrete used had the proper plasticity—I do not mean consistency—and was mixed right, the excess water would not be apparent on the surface and scaling would not occur. **Mr. Goge.**

Experience with a Strength Specification Contract.

By Robert C. Johnson.*

In July, 1926, construction was started on a large factory addition in the middle West. There were no material dealers or labor unions in the community so all material and labor required for construction work had to be brought in from other communities. The factory site being located close to the west shore of Lake Michigan has a very changeable climate, and cold weather may be expected any time from early October to April. The fall months are usually very rainy.

Construction work was divided into two units, the first consisting of two kiln foundations 400 ft. long by 50 ft. wide and two tunnels 125 ft. long by 40 ft. wide by 20 ft. deep. Plans required 10,000 cu. yd. of excavation and 6,000 cu. yd. of concrete divided up between 1: 2: 4, 1: 3: 5 and slag. The second unit consisted of two buildings, one 940 ft. by 200 ft. one story high, and the other 340 ft. by 80 ft. five stories high. The plans called for 150,000 sq. ft. of ground floor slab and 118,000 sq. ft. of reinforced flat slab. The five story section had a 22-ft. basement and floating foundations; 40,000 cu. yd. of excavation and 16,000 cu. yd. of concrete were required for this work.

Early in July representatives of five construction companies were called into consultation at the owner's office and shown a set of plans covering the kiln foundations. After being allowed fifteen minutes to check roughly over the drawings, bid forms were submitted to the contractors and they were requested to fill in unit prices for labor opposite the estimated quantities shown on the form. It was explained to the bidders that the owner would purchase the materials, except form lumber and nails, which were to be included in the unit prices submitted by the contractors. The Immel Construction Company was 50 per cent low on their proposal which was accepted at once. As the plans were not complete for the building unit, the Immel Construction Company immediately submitted unit prices covering labor for foundations for this unit and again later submitted unit prices for all concrete work involved in the superstructure. The units were accepted and contracts were signed for all of the concrete work involved in the construction program. Bids were

* Immel Construction Co., Fond du Lac, Wis.

then taken by the owner for cement, sand, gravel, and reinforcing steel. As the prices of the materials are entirely standardized in this territory, the Immel Construction Company bid a few cents under the market to obtain orders for all materials. This was done to guarantee a continuous flow of materials to the job so there would be the minimum amount of lost labor. The construction company now had contracts with the owners involving unit prices for labor on the kiln foundations, additional and different unit prices for labor on the building foundations, and a third set of unit prices for labor on the superstructure making altogether 95 separate figures not including the prices for materials f.o.b. cars. To simplify the checking of materials and labor it was decided to change the cement, sand, and gravel prices to prices per yard of concrete and combine these with the labor units. Taylor & Thompson's Tables of quantities based on 45 per cent voids were used for this purpose. Having arrived at prices per cubic yard of concrete in place for different mixes, representatives of the Immel Company explained to the owner that material had been sold at a loss of about 40¢ per yard of concrete and asked that the specifications be changed from 1: 2: 3 mix which was too rich for a 2,000-lb. concrete to a 1: 2: 4 mix. This was done and the yard price for materials was reduced. As labor prices remained the same, the contractor picked up a saving by this change. It was then proposed by the construction company that they be permitted to use any mix that would produce a 2,000-lb. concrete for the 1: 2: 4 mix or a 1,500-lb. concrete for the 1: 3: 5 mix. This was agreed to and clauses were inserted in the contract releasing the construction company from the architect's specifications but penalizing them the amount of the saving in material on any yardage of concrete that might test below 2,000 or 1,500 lb.

Very little leeway was left for the selection of materials as the brand of cement to be used was specified by the owner and the low price of local aggregates controlled the supply of sand and gravel during the warm months. After the local pits froze up, it was necessary to obtain aggregates from any available stocks. As average conditions prevailed among the local gravel pits, it was necessary to check carefully the grading, as none of the pits were used to working under a rigid specification.

Since this type of contract was highly experimental with the construction company, expense was the controlling factor in the selection of equipment. The only new machines purchased were very inexpensive weighing hoppers. Because of the large area covered by the foundations and buildings it was decided to have centralized control of the proportions with small mixers distributed over the job. This greatly simplified the work of the inspectors and engineers. Immediately upon receiving the contract the construction company hired Mr. Timms of the Portland Cement Association to take entire control of the proportioning of mixes and to be entirely responsible for strengths. The general superintendent was given a clear understanding of the importance of the strength of the concrete and

was made to realize that Mr. Timms was in absolute control of the proportioning of the material. The superintendent's greatest problem was to iron out differences between Mr. Timms and the concrete foremen who objected to the new methods. A responsible material clerk was placed in charge of the delivery of materials, the checking of quantities used, and the figuring of the units of work completed. Mr. Timms was the only extra employee required for this type of contract.

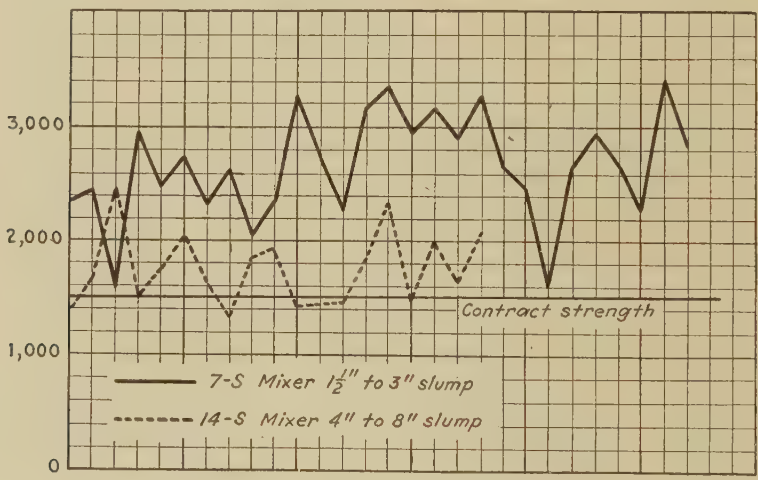


FIG. 1.—TEST RESULTS ON CONCRETE OF DIFFERENT SLUMPS AND HANDLING METHODS.

The various factors affecting the strength of the concrete at 28 days were considered and controlled as follows:

1. Cement—Quantity checked by weighing hoppers.
2. Water—Quantity checked by automatic tank, and slump test.
3. Mixing time—No rigid control.
4. Temperature—Heating before and during mixing and after placing.
5. Test methods—Not carefully checked at start of job. Later corrected.

The following methods of handling material and concrete were used: Cement, sand, and gravel were all received on cars at a siding about 1,500 ft. from the job. Cement was unloaded into a cement shed of about three-car capacity. Sand and gravel were unloaded by clamshell into bins and stockpiles. During freezing weather steam nozzles were used to thaw out the cars and to keep material from freezing in the bins. A large

marine boiler supplied steam at this location. Aggregates were weighed into batch dump trucks, the cement being placed on top. On very windy days this resulted in a loss of some cement before the material arrived at the mixer. Materials were handled in various ways at the different mixers. One 7-S mixer used for pouring ground floors was continuously moved around. The aggregates were hauled to this mixer during bad weather or while the 14-S mixers were not working. This material was stockpiled and delivered into the mixer by wheelbarrows. One 14-S mixer used for pouring the kiln foundations received pre-mixed material delivered to the skip. Another 14-S mixer was set at the base of a hoisting tower and poured

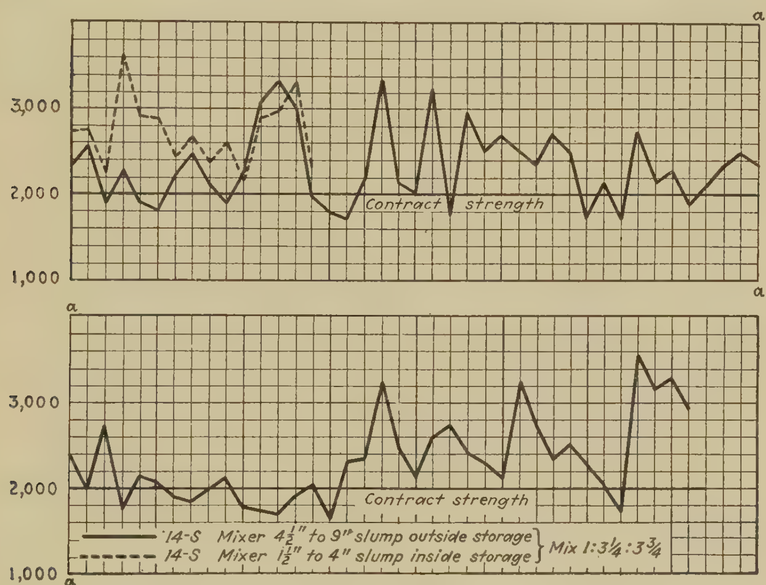


FIG. 2.—RELATION OF ACTUAL TO DESIGNED STRENGTH.

2,000-lb. concrete delivered dry mixed by the trucks. As these two mixers were pouring different proportions, it was necessary to have the drivers of the trucks wear different hats so the man at the weighing hoppers could tell which mix to weigh into the trucks. Material leaving mixers was also handled in different ways. The one-bag mixer usually dumped into wheelbarrows or direct into place. The moving two-bag mixer dumped either into two-wheel carts or directly into the forms. The stationary two-bag mixer delivered all material into a tower bucket dumping into a tower hopper and chutes. Part of this concrete was delivered from the chutes direct into forms and part into a floor hopper and two-wheel carts. During freezing weather an additional boiler was used at the stationary two-bag

mixer to heat mixing water and to run a steam line down the chutes which were entirely enclosed.

Various test strengths were obtained due to the variation in the methods of handling materials both to and from the mixers. The one-bag mixer which had no control except the slump test gave results as follows: Slumps varied from 1½ in. to 3 in. with strengths from 1,600 lb. two samples to 3,400 lb. two samples, 24 samples between 2,200 and 3,100 lb. Most of the material handled by this mixer was to pass a 1,500 lb. test. (See Fig. 1.) The moving two-bag mixer which had only weighing and slump test control gave the following results: Slumps varied from 4 in. to 8 in. and the strengths from 1,300 lb. one sample to 2,400 lb. one sample, 17 samples 1,460 lb. to 2,300 lb. (See Fig. 1.) The largest volume of concrete passed through the stationary two-bag mixer with the following results: Slumps varied from 4½ in. to 9 in. averaging between 6 and 7 in.; 90 per cent of all samples were above 1,900 lb., 80 per cent of all samples

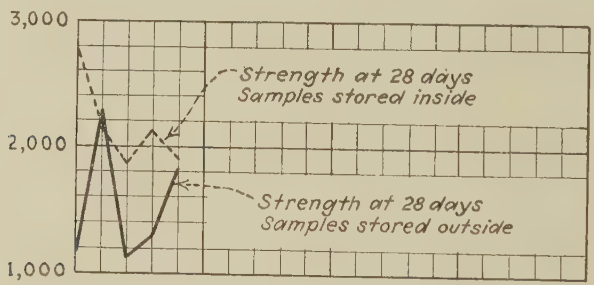


FIG. 3.—STRENGTH COMPARISON OF FIVE-COLUMN TEST SAMPLES STORED INSIDE AND OUTSIDE.

were above 2,000 lb., 50 per cent of all samples were above 2,200 lb., and 30 per cent of all samples were above 2,500 lb. The average of 76 samples was 2,280 lb. (See Fig. 2.)

Only 7 of the 76 samples fell below 90 per cent of the designed strength. Seventy per cent of all samples were between 1,900 lb. and 2,500 lb. All test samples as recorded above were stored under job conditions. The greatest variations in strength were caused by the temperature in storage conditions. Of 10 test columns made from five samples, five were stored inside and five outside. The samples inside averaged 620 lb. stronger at 28 days than the samples stored outside. (See Fig. 3.)

It was also found that numerous samples were improperly stored at the laboratory after being shipped from the job. After this condition was rectified, there was less trouble with low strengths. As a comparison of the strengths obtained on this contract and those which may be looked for under similar conditions, the results may be compared with the Portland Cement Association Building on which 13 out of 34 samples fell below 90 per cent of the designed strength whereas as noted above only 7 of 76 samples fell below 90 per cent of the designed strength on this contract.

A subsequent contract for the same company started in January and completed in February under the same units and same specifications showed the following results: Sixteen samples were taken having an average strength of 2,707 lb., none fell below 2,000 lb. This concrete was poured with the same mix as the rest but the samples were all stored inside. (See Fig. 2.) As an interesting side-light and to settle the question as to whether our concrete was being mixed long enough the average mixing time being 45 seconds for the stationary two-bag mixer, 5 samples of concrete were taken at the mixer and 5 samples from the same batches were taken from the floor hopper. The 5 samples from the floor hopper averaged 435 lb. stronger than the samples taken at the mixer showing that there was considerable justification for our claim that the handling of the concrete between the mixer and the forms resulted in additional mixing

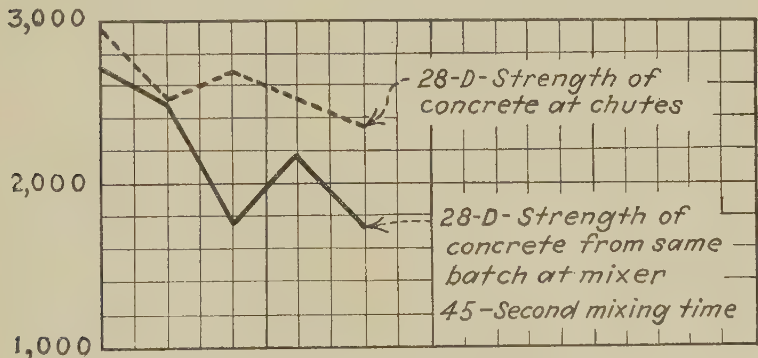


FIG. 4.—STRENGTH COMPARISON OF FIVE SAMPLES AT MIXER AND AT FLOOR HOPPER.

equivalent to leaving the material in the mixer 30 seconds longer. (See Fig. 4.)

The actual savings made on this contract were: Cement approximately one bag per cubic yard of concrete poured, aggregates 10¢ per cubic yard of concrete poured, and labor approximately 50¢ per cubic yard of concrete poured. This overcame the loss in the sale of materials below cost. Below are given computations to show the savings that may be expected from the use of a strength specification contract. For the purpose of comparison a type of plant layout has been assumed similar to that used on this contract. These computations are self-explanatory.

ANALYSIS OF STRENGTH CONTRACT ECONOMIES.

1. *Assume type of equipment lay-out*
   - Aggregate bins
   - Weighing hoppers
   - Trucks

Cement shed
Two-bag mixer
Tower and chutes
Buggies

2. *Assume Labor and Material Rates*

Cement, $2.19 per bbl.
Sand, $1.40 per cu. yd.
Stone, $1.535 per cu. yd.
Common labor, 45¢
Skilled labor, 75¢

3. *Cost of Concrete as Originally Specified*

A. Résumé of Specifications

Mix 1 : 2 : 3 — 1 bag cement = 1 cu. ft.
Slump 6 in. to 7 in.
Time 1½ min.

B. Analysis Material Costs

Cement,	1.67 bbl.	x 2.19	= 3.6573
Sand,	.50 cu. yd.	x 1.40	= .70
Stone,	.74 cu. yd.	x 1.535	= 1.1359
Total			5.4932 = 16.2 cu. ft. per bbl. 8.1 cu. ft. per batch

C. Analysis Labor Costs

1	Foreman	10 x 1.35 =	13.50
1	Clamshell operator	10 x 1.25 =	12.50
2	Clamshell laborers	20 x .45 =	9.00
1	Weighing man	10 x .50 =	5.00
1	Bin laborer	10 x .45 =	4.50
2	Cement men	20 x .50 =	10.00
3	Truck drivers	30 x .50 =	15.00
1	Mixer engineer	10 x .75 =	7.50
1	Mixer laborer	10 x .45 =	4.50
1	Hoist engineer	10 x 1.25 =	12.50
1	Tower man	10 x .60 =	6.00
5	Buggy men	50 x .45 =	22.50
3	Spaders	30 x .45 =	13.50
			146.00
	Liability Insurance		7.30
	Total		$153.30 per 10-hour day

Allow 2½ min. per batch = 240 batches per 10-hour day
240 x 8.1 = 1,944 cu. ft. per day = 72 cu. yd.

$$\frac{153.30}{72} = \$2.13 \text{ per cu. yd. labor}$$
5.50 per cu. yd. material

$7.63 per cu. yd. total

4. *Cost of Concrete as Poured*

A. Analysis Material Costs

Cement,	1.25 bbl. cement	x 2.19	= 2.7375
Sand,	.56 cu. yd. sand	x 1.40	= .7840
Stone,	.74 cu. yd. stone	x 1.535	= 1.1359
			4.6574

21.7 cu. ft. per bbl.
10.85 cu. ft. per batch

B. Analysis Labor Costs

Same crew as above	= $146.00
Mix engineer	= 14.00
	$160.00
Liab. Insurance ..	8.00
Total	$168.00

Allow 1¾ min. per batch = 343 batches per 10-hour day
343 x 10.85 = 3721.55 cu. ft. per day = 137.83 cu. yd.

$$\frac{168.00}{137.83} = 1.22 \text{ cu. yd. labor}$$
4.657 cu. yd. material

$5.88 cu. yd. total

Specified concrete cost	= $7.63
Contract concrete cost	= 5.88

Saving .... $1.75 per cu. yd.

Part of this saving was given to the owner by changing basis of contract from 1: 2: 3 mix to 1: 2: 4 mix so it represents more than the total saving per cu. yd. as made by the contractor.

As a comparison of the results that might have been expected for the concrete as originally specified and for the results as obtained, there are unfortunately very little test data. However, during the months of February, March, and April a different organization poured concrete for a superstructure for which we had poured the foundations in January. The superstructure was built in accordance with the specifications as originally drawn for the pottery unit. Four samples of this concrete were taken, the

methods of sampling and storing being the same as for the foundations. The average of these four samples was 2,495 lb. whereas the average of the 16 samples taken from the foundations poured under the strength contract was 2,707 with none falling below 2,000 lb.

To obtain the maximum economy under any strength contract, curves should be worked out for the plant setup and the most economical ratio should be obtained for mixing time, labor cost and material cost. A short mixing time necessarily means more cement and less labor, so the most economical and practical ratio between the material and labor costs should be determined for each job. It would be proper to take five samples with five variations of cement content for each of the following mixing times: 45 seconds, one minute, one minute and 15 seconds, one minute and 30 seconds. In running these job control tests the water-cement ratio should be maintained practically constant. Having obtained the results of these tests it is a simple matter to determine the most economical mix by comparing the total cost of concrete per yard for each mix based on an average daily output for each mixing time. It is the opinion of the writer contrary to the belief of the average contractor that the most economical ratio is obtained with one minute and 30 seconds mixing time.

Conclusions.—This particular contract was naturally very satisfactory both to the owner and to the contractor. A concrete of satisfactory strength was furnished the owner at a cost less than would have been required by following the specifications whereas at the same time the contractor had absolute control of the methods of manufacture. There was a greater spirit of co-operation between the inspectors and the contractor's organization in trying to obtain a given result than there would have been had the inspectors been merely trying to enforce a rigid specification. It was quite apparent that there are no definite and accepted standards for the measurement and manufacture of a given strength concrete. Where a spirit of co-operation is built up between the designer and manufacturer, there is not as much reason to worry about the quality of concrete as there is if a continual struggle exists to enforce in some cases an unreasonable specification.

There are economies of design and construction which will make the strength manufacture of concrete a necessity of the future. To meet this situation a committee of the American Concrete Institute should clarify the following points:

1. What standard should be used for the measurement of concrete strength.
2. What over and under-run should be permissible due to the methods of testing strength.
3. What is an equitable form of contract to guarantee results.
4. What would be the most economical methods to use in the production of strength concrete.

The question about strength of concrete today is not whether we want it but how can we handle it?

Discussion.—Experience with a Strength Specification Contract.

Joseph A. Kitts.*—This paper brings out several points of interest and suggestions of value to the concrete industry. It is a record of success with the strength specification contract both in quality of concrete obtained and economy effected. Mr. Kitts.

The quality (strength, etc.) specification is coming more and more in favor. As knowledge of concrete becomes more general, there is no doubt but that the quality specification will become the general rule. Of 660,000 cu. yd. of concrete production controlled by the writer and associates in the last three years, 640,000 or 97 per cent was quality (strength, weight, density) specification concrete as compared with 3 per cent to arbitrarily fixed proportions.

To specify the quality required is logical, practical and scientifically correct economically and technically. It is the best means to promote efficiency in production, to encourage research and to increase and reward a knowledge of concrete.

The author shows that one man-hr. of control for every 13.8 cu. yd. effected a saving of $1.75 per cu. yd. (including the cost of control) as compared with the cost of usual arbitrary proportions. A saving of one-tenth this amount would have been a good investment in control of concrete production. For a rate of production of 138 cu. yd. per day, the yardage per man-hr. of control is at an economical ratio under favorable conditions. Where the rate of production is 1,000 cu. yd. or more per day, the economical yardage per man-hr. of control will vary from 25 to 50, depending upon the conditions.

The effects of varying ratio of yardage to man-hrs. of control have been determined by the writer in connection with the Exchequer Dam project of the Merced Irrigation District, California, on which the production averaged about 1,000 cu. yd. per day. The results are shown in Table I.

The high strength to cement ratio as shown in the table is due to the large size of the aggregate used. Four sizes were used,—sand, fine gravel, coarse gravel and cobbles. The practical maximum size was 4½ in. (square opening screen). About 3 per cent of the aggregate was from 4½ in. to 9 in. The aggregate proportions were the same in conditions (A) and (B) and the difference in the "economy factors" are no doubt largely due to

*Consulting Concrete Technologist, Kitts & Tuthill, San Francisco.

the relative efficiencies of the cement contents. The greater "spread" of (B) is no doubt partly due to the greater period of testing and the least "spread" of (D) likewise to the comparatively short period of tests.

In the cases of control conditions, A and B, where one man-hr. of control was employed for every 120 yd. of output, all that could be accomplished was the testing of the quality of the aggregates and concrete as produced. For the C condition of 80 yd. output per man-hr. of control,

TABLE I.—RELATIONS OF STRENGTH, STRENGTH SPREAD, STRENGTH ECONOMY FACTOR AND CEMENT CONTENT TO OUTPUT PER MAN-HR. OF PRODUCTION CONTROL.

Yardage Output per Man-hr. of Control	Minimum Average Maximum and Spread of Strength at 28 days	True Mix and bbl. Cement per cu. yd. of Concrete	Cost of Aggregate Cement and Control per cu. yd. of Concrete	Economy Factor, Strength per Dollar Cost of Materials and Control	Type of Control and Number of Days Tested
120	1435 1920 2750 1315	1:7.8 0.843	1.830 2.529 0.015 4.374	328 439	(A) Arbitrary proportions. (33 days)
120	1075 1650 2535 1460	1:9.2 0.720	1.886 2.160 0.015 4.061	265 407	(B) Arbitrary proportions. (71 days)
80	1445 1910 2735 1290	1:8.4 0.784	1.850 2.352 0.022 4.224	342 452	(C) Arbitrary proportions, improved aggregate. (71 days)
60	1755 1865 1985 230	1:9.2 0.719	1.850 2.157 0.029 4.036	435 462	(D) Approximately controlled grading and improved aggregate. (9 days)
40	1880 2110 2310 430	1:8.6 0.764	1.850 2.292 .044 4.186	448 503	(E) Accurately controlled grading and improved aggregate. (23 days)

the quality of the aggregate could be followed more closely and some improvements in the quality of same effected. For the D condition of 60 yd. per man-hr., it was possible to follow the variations of moisture content, bulking, density and fineness modulus of the four aggregates so as to maintain approximate uniformity in the grading of the mixed aggregate. Under the E condition of 40 yd. per man-hr., it was possible to maintain uniformity of grading with a measure of accuracy.

Under like conditions of cement content, the strengths and strengths per dollar cost of materials and control for the comparative conditions of yardage output per man-hr. of control, are as shown in Table II.

Table II shows that 20 per cent greater strength in proportion to cost was obtained under accurate control conditions, and the results indicate that maximum economy is in the direction of still more control. Starting with a concrete costing $4.374 per cu. yd. (including control) a uniform and satisfactory concrete was made at $4.036 by doubling the control, thus effecting a saving of $0.338 per cu. yd. or $338.00 per day.

The compression test results obtained by the author indicate that the strength of the concrete fully met the specifications. The small percentage of tests below the specified strength are no doubt due to the law of probability and error in measurement, particularly to the usual adverse errors of compression tests, and do not indicate the true strengths of the specimens. This points to the importance of the mooted points 1 and 2 of the author's conclusions, which seem proper subjects for discussion in this connection.

TABLE II.—CONTROL RATE, CEMENT CONTENT, COST, AVERAGE STRENGTH AND STRENGTH ECONOMY FACTOR.

Yardage Output per Man-hr. of Control	Cement, bbl. per cu. yd. of Concrete	Cost of Materials and Control per cu. yd. of Concrete	Average Strength of Concrete, lb. per sq. in.	Strength per Dollar Cost of Materials and Control
(A) 120	0.843	4.374	1920	439
120	0.784	4.223	1790	424
(C) 80	0.784	4.224	1910	452
(B) 120	0.720	4.061	1650	407
(D) 60	0.719	4.036	1865	462
120	0.764	4.173	1750	419
(E) 40	0.764	4.186	2110	503

Low strengths are indicated by damaged, defective and erroneously measured specimens, by specimens with concave, convex, oblique, and otherwise defective caps, by specimens not centered in the compression machine, by samples not representative of the concrete in the structure and by specimens not properly cured. Harrison F. Gonnerman shows that slight defects in the end condition of cylinders give measured strengths as low as 40 per cent of the strength of true and accurately measured specimens of the same concrete. (See Bulletin 14, Structural Materials Research Laboratory, Lewis Institute, Chicago.)

Messrs. Slater and Walker, in their "Report on Field Tests of Concrete Used in Construction Work," submitted to the Joint Committee on Standard Specifications for Concrete, suggested the average strength as the "design strength" with a tolerance of 10 per cent of measured specimens falling below 80 per cent of the average measured strength at 28 days. (See *Proceedings*, Am. Soc. C. E., January, 1925.) On this basis it would be a logical rule to determine the "design strength" as that measured strength (not greater than the average strength) at which there are not

more than 10 per cent of specimens less than 80 per cent of that strength. As an example, if 10 per cent of specimens were measured as 1,600 and less, the design strength would be 2,000, provided the average measured strength was not less than 2,000 lb. per sq. in. at 28 days. This does not assume that the actual strengths are as measured but that, in all probability, due to the established adverse errors of compression tests, the average measured strength (with the tolerance given) represents the actual minimum strength. Any uncertainty is provided for by the usual safety factor of structure design, and also, before the structure receives its maximum load (allowing an age of three months), the concrete will have increased at least 25 per cent in strength over that of the 28-day age. No retrogression is likely from the strength at the age of 3 months.

We find, under accurate and economical production rate control and fairly standard conditions of curing and testing, that not more than 10 per cent of specimens fall below 90 per cent of the average measured strength. Under the usual conditions of curing and compression tests, 10 per cent below 80 per cent of the average is a satisfactory tolerance.

We all have approached concrete specifications from the point of view that the proportions of the ingredients should be fixed as so much cement, sand and rock. Recently, the ratio of the ingredients water and cement has been proposed as the basic specification. Undoubtedly, the quality required is and should be treated as the fundamental basis of the specification covering the concrete manufacture.

The usual quality specification differs from the standard specification only in that it provides for competent technological control of the concrete manufacture and a separate form of payment for the cement used. The engineer or architect representing the owner usually assumes the responsibility of control and results. Where the responsibility for results is put upon the contractor, mutual control is maintained or the contractor has his own control and an independent check is maintained by the contractee.

I am firmly convinced that the best means to further the interests of the concrete industry, to promote efficiency in concrete production, to encourage research, and to increase and reward a knowledge of concrete is to establish a quality (strength, etc.) specification as the standard specification, with appreciation of the facts that concrete production is a manufacturing process, that the manufacture of concrete of quality with economy requires thorough, constant and understanding technical and practical control of the proportioning, measuring, mixing, placing and curing, in co-ordination with and supported by, cement, aggregate and concrete testing as a daily routine part of such production control. I strongly second the author's recommendation that a committee of the American Concrete Institute work upon these questions now before us.

Mr. Slater. W. A. SLATER.—It seems that the engineers are going up one elevator while the contractors are going down the other, and then the contractors go up and the engineers go down; they do not seem to get together on the same floor. A few years ago when the Joint Committee was making speci-

fications, they made two alternate specifications on the same subject; one of them was a proportion specification and the other was a strength specification. They thought when they proposed the alternate strength specification that the contractors who wanted to get their money's worth and deliver a good job, would hail it with open arms. But the contractors did not and neither did a good many other people. From the 2 or 3 days of discussion that we had at the meeting of the American Concrete Institute and of the American Society of Civil Engineers, it certainly appeared that the contractors were not unanimous on the strength specification. Finally the strength specification was withdrawn. It seems to me there are some indications that the contractors are now ready for it, or is it not more than one contractor?

Better Concrete.—Do We Mean It?

By Nathan C. Johnson.*

The concrete industry in the United States calls for the annual expenditure of approximately $2,000,000,000 by those who are sold on concrete. Two billions of sales per year for any commodity is an achievement that must be based on real value and real merit.

Two billions of dollars per annum for concrete in the United States alone indicates and proves an unquestioned value and utility for concrete. But two billions of dollars is not enough. Four billions of dollars would be a more normal sales demand for the needs of the present age in constructions that would employ concrete if a departure were taken from the idea that any "mud" that contains cement and fills forms is properly classed as "concrete." If a better understanding were had of the innate and intrinsic values and virtues and abilities of real concrete, four billions of dollars of demand as of 1930 would not be a fanciful dream, but would more likely prove an estimate falling short of the actuality.

But if we would progress in the use of concrete and create this four-billion demand by 1930, we must focus our energies, our time, and our efforts to make concrete meet every reasonable requirement. And by "reasonable" is meant every requirement *falling within the natural properties and abilities of concrete.*

The significance of that statement is not generally appreciated, nor can it be hoped that there will be appreciation of the vast and almost illimitable abilities of concrete until the majority, both professional and lay, are not so much taught, as shown by example what may be done with this Protean material. Not in the old way, for that way is founded on such gross misconceptions and misunderstandings as to seriously limit the utility and usefulness of concrete, but in a new and better way, founded on real knowledge as to what are and what are not the full, natural abilities of concrete.

The burden of this research and this demonstration must be assumed by some one or some group, and it will take relatively little money but worlds of courage and plenty of that quality that is best described by the robust term of "guts" to blaze this trail. There will be found plenty to follow where the path has been opened.

No class or individual is exempt from this call to service; and each has today an opportunity for achievement in this field which is without parallel in any other industry.

*Hool and Johnson, Consulting Engineers, New York City.

This paper, therefore, will be not so much a presentation of minute and controversial technicalities, as a plea for a broader viewpoint, for a better and broader knowledge by all connected with the industry, or who utilize the industry. And it will be particularly a plea for the suppression of "isms" of one kind or another which bring only opposition and create confusion where there should be united purpose.

To this end, let us mutually and good-naturedly make an examination of the art as it is today, that we may either strike a balance by this audit, or else fail to strike a balance.

Since commerce and the commercial use of concrete provide the life-blood of the industry, let us examine what happens when a projected commercial structure starts actively to come into being.

Assume that the structure is a dock or pier to be built in some harbor basin that is filled and emptied more or less twice a day with sea-water through the increase and recession of the tides. The allowable cost is 2½ million dollars; and commerce demands completion in eleven months. Engineers and/or architects start the ball rolling. Concrete is the structural material chosen beyond a doubt. And inasmuch as we are here concerned with concrete and its usages and behavior, auxiliary data, such as depths of water, soil conditions, rail connections on the landside, etc., etc., can be neglected as of this analysis.

First of all, a dock or a pier, is a one-story or a two-story concrete building with a flooded basement. No more, no less. Any long concrete building, flooded inside and out up to an assumed mean-water level, would be, in essence and in most design and structural details, a pier. The structural design problem is therefore the same as of any building—almost a routine operation today.

The construction problem would be one of floating equipment versus landborne equipment but employing essentially the same machinery; and of supporting piles as against caisson or spread footings. The deck is a simple floor. The pier shed is no problem. The concrete throughout is probably either 1: 2: 4, or a theoretical mix; the design and the stresses are taken from a handbook; and the supreme art embodied in the job will probably be that of jockeying to land the contract and of making money on the contract when it is landed—no mean task.

But what of the concrete problem—to produce for this costly structure a concrete that will endure for, let us say, 25 yr. without sensible deterioration? How shall it be done? How shall we as engineers, or you as contractors, or they as owners know of a surety when the retained percentage is paid off, that the qualities necessary for such endurance are embodied in the structure? Shall we be honest in answering these questions, or shall we take the viewpoint of the little girl who rebuked her mother's mention of "toilet water" by saying: "That, Momsey, is something that nice people never speak about"?

This is an audit, so let us be honest. When this concrete was in the making, or when it was made, there was no way, no real knowledge, no

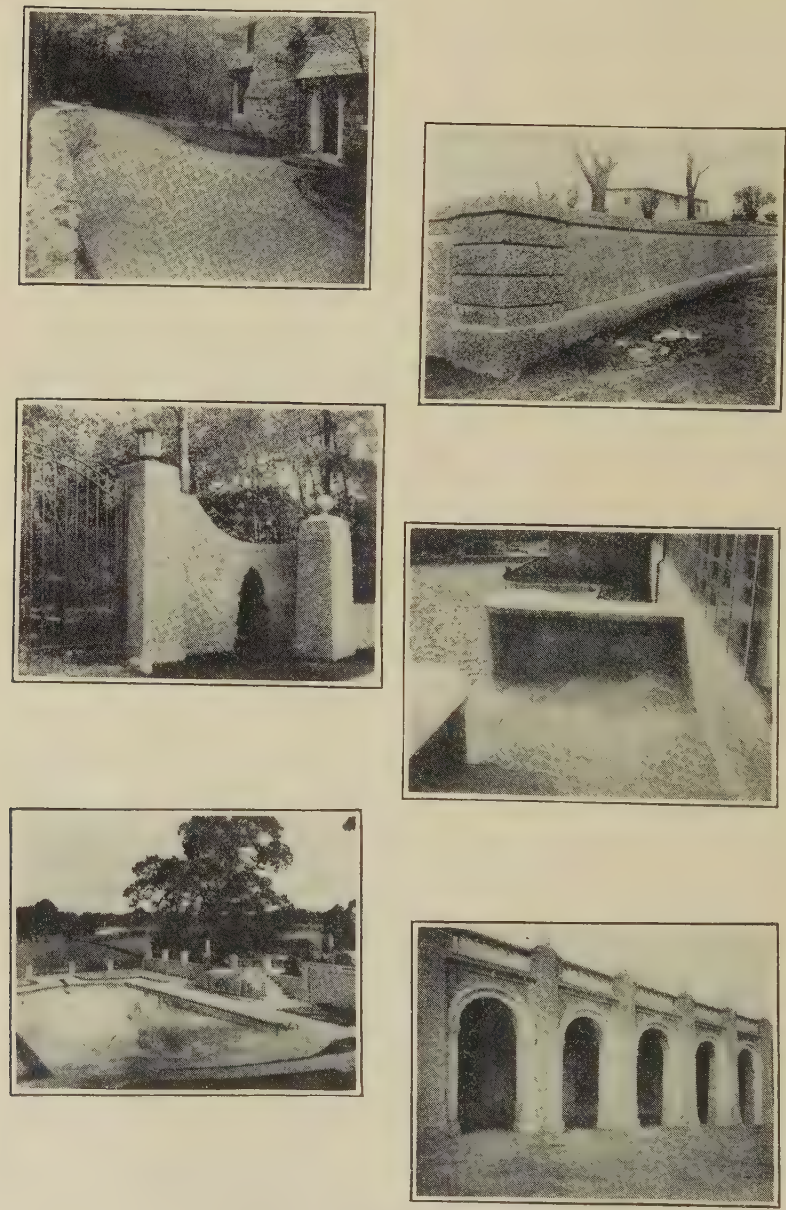


FIG. 1.—THE REALIZATION OF ACCEPTABLE ARCHITECTURAL APPEARANCE.

means at hand, even approximately to determine the value of that material in vital members or its endurance under water exposure. Strength test cylinders may have been made as the work progressed and may have proven satisfactory. But what has compressive strength to do with endurance under water exposure? Nothing, so far as we know. And of how much value as a protective agent is some outside coating—such as, perhaps, Himalayan Black Jack? No one knows, except that it is black. And what if the purchasing agent saved 15¢ a gallon by substituting Peruvian Pitch for Himalayan Black Jack? Both were equally black, even though the Peruvian Pitch proved later to be quite soluble in water. Both were black.

Assume further, as may occur, that the concrete in this pier does soften and go to pieces more or less after a year or two, why does this happen? What properties were lacking in the concrete itself—properties that were necessary, that should have been measurable, and being measurable should have been producible and reproducible by the artisans and engineers of this proud, scientific age? And most curious of all, why did some of the concrete go bad and not the rest, when all concerned are ready to assert on oath and with full faith that all work was done in a thorough and workmanlike manner, in full accordance with the specifications and the design and under the supervision and approval of the great Dr. Blank himself—an authority on concrete without a peer?

Embarrassing questions and not fiction. To hit the four-billion bull's eye, answers must be found to these and to many others as well. Nor is the random example chosen an exaggeration. Essentially all the conditions therein noted are arising daily on thousands of other types of constructions in concrete. The only condition therein assumed that is not universal is the condition of water, exposure, and assumed water protection of the concrete by "guaranteed-by-the-manufacturer," additions of soap or of oil to the mix, or by outside coatings.

As to sea-water concrete, a yardstick essential to measure the qualities that give endurance seems to have been developed, and more accurate knowledge than heretofore has been had is in process of derivation and assembly. It promises well for constructive additions to our knowledge of concrete, and we must have this knowledge in the near future or surrender the field and the market to other materials.

Corrosions are interesting things and valuable as a study from which some of this necessary and valuable knowledge may be gained. Out of any study of corrosions, curiously enough, will come a new respect for concrete and for its integrity. Any ardent corrosionist would gain in wisdom and in knowledge as well as in prudence of speech if he were to essay to disintegrate even a cubic foot of medium-poor concrete. Out of that ordeal, even were he furnished with all the armory provided by modern chemistry, with all its powerful reagents, he would emerge a sweaty and chastened pupil, far more willing to talk of the damnable refusal of concrete to go to pieces than to speak largely upon the subject of its perishabilities.

But since we have entered upon this audit and this game of "Truth and

Consequences" let us go a step further while on this subject of the good and the bad in concrete.

Given six experts from field, office or laboratory and require of them a supportable diagnosis of some trouble, or of some perfection, it is probable that no two will today agree on essential causes, because concrete, as a material, while a familiar thing, is also one virtually unknown and none will be able to support their individual opinions except by virtue of personal magnetism of manner, by effrontery unbacked by knowledge, or

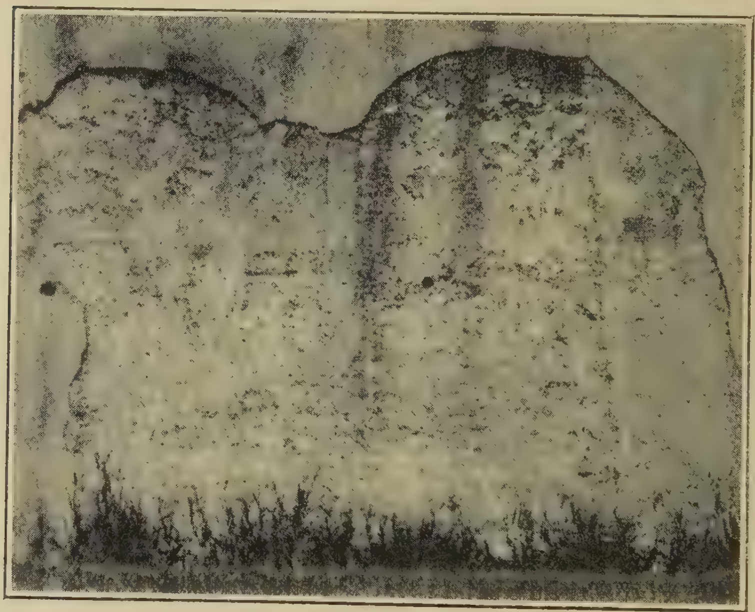


FIG. 2.—A BOND THAT ALWAYS FAILS—THOROUGHLY HACKED CONCRETE AFTER 3½ YEARS.

by an excessively large tummy and a profound and unpuncturable dignity that too often has proven of more weight than real knowledge.

Sadly enough, concrete *per se* is an unknown land to the majority. Design in concrete is understood, because design deals in figures and in pictures and in blueprints that are understandable, that are tangible, and that may be seen and touched, and that may be evaluated *per se*. Handling concrete as an art of handling materials, is understood, because there is money in rapid handling, and yardage is measurable and money is measurable. But the very heart of the whole thing—the concrete itself—is not even passably understood by many, regardless of how willingly they tell us all about it and how vehemently they assert that they are "practical

men." It often seems as though the "practical" man were the greatest optimist in the world, for he seems to have faith that if he tries the wrong way often enough, some day he will succeed in reversing Nature and make good on his daily and prideful boasts.

There are, then, yardsticks applicable to these incident phases of the concrete art. But all that is so beautifully pictured on tracing-cloth is not actually made in the field, and the alibi for haste and carelessness in the field is test cylinders for compression strength alone. These cylinders are

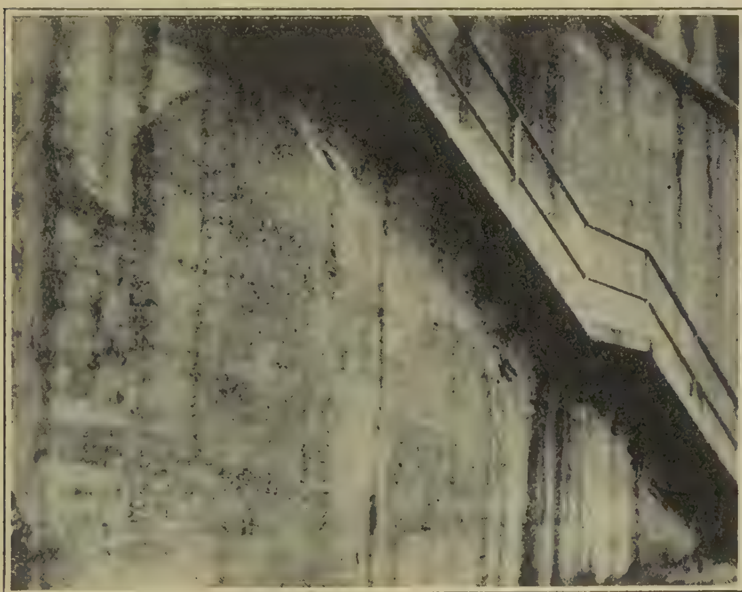


FIG. 3.—A FAILING BUSH-HAMMERED SURFACE.

Lacking in distinction in bush-hammered areas with the usual staining of the floated form skin in the plain areas. Faulty placement of concrete beginning to show in stratification.

tangible, but accepted (even on adverse results) with so great a tolerance that any rejection of concrete in forms on the basis of tests, even where this concrete is undeniably, visibly, and grossly below acceptable quality, is unknown to the writer. Instead, visibly defective concrete is smeared over with grout and then called "perfect"—"and if you don't believe it is perfect, see our Company's attorney."

So it has come about that when something goes wrong, there is no means of quality determination or predetermination in general acceptance which will teach an offending engineer, or contractor, or superintendent, or foreman, or laborer, or cement-maker, or laboratory-man, or supply-man

the facts in the case, so that little by little as the years go on, a material of full dependence shall be produced, until, whenever "concrete" is called for, that call will be met automatically and as a matter of course by the production of material having full reliability.

If we are to create a larger and better industry, means of true and accurate measurement must be evolved. Some forty years ago the quantity production of electricity was brought about through finding avenues of sale for that electricity in the production of light and of power. The art as we know it and have come to depend on it was then crude, but it has grown and is still growing to proportions beyond the imagination, because hand in hand with quantity production went the evolving of means to measure that production. Only two days ago radio television was publicly demonstrated to fillip anew our senses and to stimulate our buying activities. If we show the accomplishment, the buyers will come in droves. It has been proven over and over again.

There may or may not be significance to us in the fact that all these means of measurement that have brought about the growth and development and purchase by the public of electrical energy have been founded on indirect phenomena. No one yet ever saw electricity. But all of us have seen a lamp glow and a motor run. We have seen the indicator of an ammeter or a voltmeter move, and a lesser number have observed other mechanical things of one kind or another indicate one phase, or property of electricity in which we are interested.

It is even more than possible, then, that if we look for indirect means of measurement, we, in our art of using cement, may learn various things that we need to know if we are to create a four-billion-dollar industry by 1930.

Obviously, since we are dealing not with one material, or with one force, but rather with combined material of different kinds and sizes, and with combined forces, we must take more pains and expend more thought than would be the case where an isolated and uniform material was to be measured.

A good starting point, then, would seem to be a study of the individual materials and later on, a study of the combination of all of the several materials.

The remarkable integrity of concrete—the astonishing exhibition of power in opposition to forces of disintegration—has before been commented on. This power comes from cement, of course, and our path may be made easier if we keep in mind, not as a definition but as a thought, that cement—the common, gray powder that comes in sacks—is a source of power. We release this power and turn it to our uses by wetting cement with water. When these two substances unite chemically, a new substance is formed that is no longer portland cement, but is a derivative material that is of a new and different nature.

Integrity in concrete, then, is due primarily to that peculiar and somewhat temperamental substance that is formed when cement and water

rest together for a sufficient time in undisturbed and peaceful union. To hit the four-billion mark, we of course ought to know more about cement and the manufacture of cement, but we ought also to know more about this substance that is derived from cement plus water, for that is the "glue" that sticks together the sand and the stone put with cement and water to form the cheap and useful substance called "concrete" when there is stone present, and called "mortar" when there are no mineral particles present over ¼ in. in size.

FIG. 4.—CUSTOMARY APPEARANCE OF STUCCO OVER THE FORM SKIN OF CONCRETE AFTER A YEAR OR SO OF EXPOSURE.

Investigation of how to make cement, of how to make tricalcium silicate or dicalcium silicate, predominate in the product of these great synthetic processes that are employed in modern cement mills today, is a whole and distinct field in itself. The compass of this paper will not permit any discussion of this, even were the writer abreast today of all modern scientific investigation in regard to it.

But this all of us know: that so far as the great cement-consuming public is concerned, cement is one brand or another of a gray powder that comes in sacks; and that this powder wetted with water and allowed to stand, will stick sand and stone together in a wonderfully useful mass; and that all this will happen, whether the mixture is made by a learned professor, or by a laborer with a mediaeval mental outlook. We all know,

too, that strength results from this union—strength to bear load, strength to do this task and that task for us; and that money is to be made in putting this mixture into places where other people want bridges and buildings and dams and roads and reservoirs, and so on.

We all know, too, that recently we have been somewhat mixed up about this powder, because certain makers tell us that their brand of powder is twice as good as another brand, because some laboratory has found that a month after any sand and stone and water are mixed with it, some nice little rollers made from it are twice as strong as other little rollers made from the other cement.

But the practical minds of all of us cannot help inquiring about the value of this. Does it means that we can strip our forms in one day instead of four? Does it mean that we shall have ten years of life in water exposure instead of five? Does it mean we can use half the quantity of cement? And if not, since the old kind of cement gave us more strength than we needed, why should we double the strength?

And as a more pertinent question we ask: Is this increased early strength really what we want? Is so reactive a cement giving up all its power at one time and having nothing left to go on in after years? Are we paying for a spectacular "explosion" as compared to a faithful and steady evolution of power? And if not,—if there is additional and new virtue in these improved cements,—is there not some way of measuring this new virtue, so that we may evaluate it and assay it and find out some way to use it, as, for instance, in finding in concrete a tensional stress that may be relied on, so that less steel may be required and our structures made unassailably cheaper in cost, and therefore find more extended use in a greater variety of structures?

This possibility alone, if realized, would help materially in knocking the four-billion ball over the fence; and we wait with interest and eagerness some information on the subject from authoritative sources.

But even though we stick to the old gray powder, with its known virtues, we can have open and inquiring minds as to new uses for it. Perhaps, also, we can devise a means of measurement that will tell us how to put quality into our concrete.

As before stated, this cement powder, with water, gives off a whitish "glue"; and this "glue" will usefully stick stone and sand together in a very satisfactory manner. Under a magnifying glass, or better, under a microscope we can watch this glue forming; and we can both see in this manner and measure in other ways, that the formation of this glue is very rapid if the temperature is high, and so very slow at about 38 deg. F. as to be almost imperceptible. So, then, we say in the field, that in hot weather forms may be removed very soon after placing concrete, and in cold weather, we must wait a longer time until the cement has set and hardened.

Obviously, for this reason alone, any means we can devise of reliably increasing this production of glue, regardless of hot weather or of cold

weather, would enable us to better compete with other structural materials and at a greater profit. This means might be a catalyzer—a substance that itself does not change in nature or enter into the reaction, but that speeds up the reaction; but thus far, no reliable and sure catalyzer has been found, but all we need to do is to hunt a little harder.

Mechanical attrition with slurry-grinding of the cement particles during the normal mixing period has been found very effective. Although not as yet commercially available to all, there is a machine which gives more

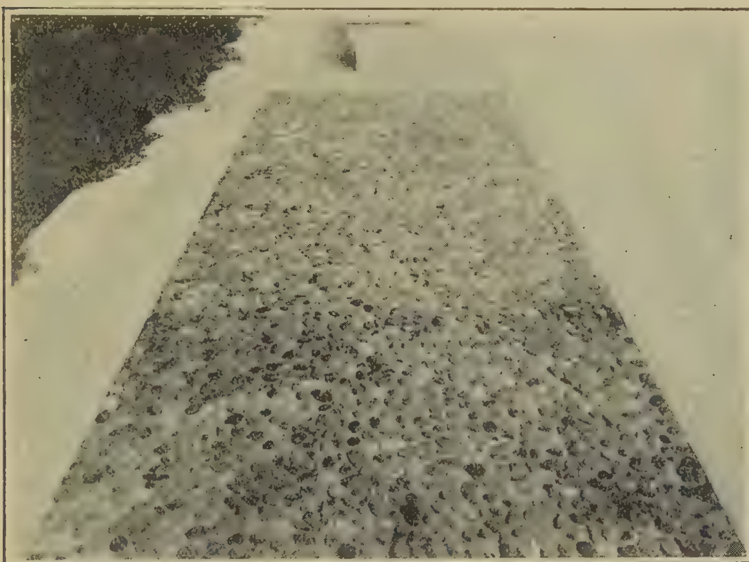


FIG. 5.—ARCHITECTURAL USES OF CONCRETE DEVELOPED BY SURFACING—MEADE MEMORIAL, WASHINGTON, D. C.

glue, more strength, and more endurance to concrete than do machines which fail of this attrition in their operation.

These are clues; and as we study the subject more and more, we begin to get more clues and even more clues as to what can be done provided we possess a little guts and courage. As we progress in this way, and as our eyes look outward instead of inward, four billions of sales by 1930 begins to look more and more like a piker's bet. Each hill climbed gives us new and ever broader horizons, and the once-distant mountain is not so far off as it was a few years ago, for we have airplanes today of more than one variety and the old lifetime trudge is now only a hop.

But while studying the evolution of this useful glue from cement and water, we must also look at other things—other things literally by the

billions. The co-partners in usefulness of this glue are sand grains and stone particles. We buy them and use them literally by the billions and, although we seldom do so, it pays us to sit with them, to get acquainted with them, and to appreciate them as we should our business associates. If we turn our quizzing-glass upon them, we find jewels of color and of form, worthy of a costly setting. But because they are common, and because they are useful, and because the Medusa on the dollar drives us, the only setting we give them is a coating of our useful and unbeautiful friend, the glue that comes from cement and water.

So, when we make concrete we bury all these colors and all these excellencies with but occasionally a pang that even the commonest of laborers will admit when he picks out in his work a superlatively lovely stone and puts it into his pocket to take home for the kids. Millions and hundreds of millions of equally beautiful ones we bury in forms with never a thought; and we have grown accustomed to having the greatest satisfaction in our work if, when forms are removed, this burial is found to be most complete by having no break in the evenness of the obscuring cement that lies over them.

But if, while we were doing this, some imperious devil of curiosity had required of us a removal of this cement shroud, so as to determine, as of a possible visual yardstick, the evenness of our mixture as it finally lay in forms, we should have had a revelation. In the first place, we should find on most work done in the past, that our mix was anything but even. We would be shocked at the difference between the even sections depicted by designers on our drawings and the pockets of stone here, the pockets of muck there, and the stratification everywhere that stood revealed for us to see. And we might very possibly exclaim to ourselves that it was "no wonder that Section 3 was going to Hades already."

Of course, we would then try again, and pretty soon we would learn how to place concrete and how to mix it and how to proportion it, so that we could fearlessly undress any of our handiwork, no matter what sort of a structure it was in.

And as we took home with us that evening the vision and image of that inspectable and respectable concrete that even our labor gang had enthused over, all of a sudden would come a realization that many things had been accomplished that had been long desired. By revealing the beauties of the burial stone, we had very naturally made the concrete beautiful—suitable indeed, if aggregates were chosen for size and for color and so on—for the finest of architectural uses.

Stone, and in maximum quantity, tooled by the glaciers of ancient days if gravel were used, or by crushers if rock were used, but reassembled by ourselves and bound together by cement as a permanent surface for our structures . . . Four billions in 1930? Easy!

And as we lived with this, we should learn a world about concrete that opened even broader horizons. The glue that we studied so long ago needed and demanded and got clean stone surfaces to stick to, else it would

not stick. It stubbornly refused to stick to its own near relations, such as the surface cement of old concrete, except for a little while; and as for permanently marrying with them, there was nothing doing.

But here, in this "undressed concrete," were clean stone surfaces in plenty. New cement glue would surely stick to them; and that meant that new concrete could be bonded to old; that mortar or stucco could be bonded to a wall; that plaster could be bonded to a ceiling; and even that members could be pre-cast in a factory and silently "welded" by a pouring of grout.

FIG. 6.—ALL-CONCRETE BUILDING, BOND SURFACED FOR STUCCO.

And as this eventually turned out, it meant other things—even to a complete new philosophy of proportioning and a new system of building that today and as of the contracts of 1927, stands concrete over 30 millions of dollars to the good and toward the four-billion goal.

Time and space do not admit of recovering much that should be in this paper. It is possible that much that is tempting of inclusion is too intricate for presentation, except through some medium which will permit of its slow and reflective assimilation by the concrete fraternity.

Progress is with us. But the great total of progress that will call upon the industry to furnish annually to the public four billions or even six billions of value within the few years next to come will be brought about by the many, not the few, and not least, by the laying aside of old

jealousies, old dicta, and old prejudices, with a joining together of all along broad lines.

But do the vast majority of concrete makers and sellers want "better concrete"? They talk of it but do they mean to get it?

The fair answer as of commerce is, that "better" in the commercial sense is closely allied to "cheaper." There is no slur in saying that "cheaper concrete" would open many a sleepy eye at any discussion on concrete.

And fortunately, "better concrete" and "cheaper concrete" are identical. Research—field research as well as laboratory research—proves this to be true. In fact, these twin results *must be acomplished* in the general run of work, for the increase of usage of cement in the past few years is not commensurate by any means with the increase of usage of competing materials.

But through the present efforts of a few, the tide seems about to turn.

Discussion—Better Concrete.

W. J. Barney.—This is a very important paper; it seems to me it should be brought to the attention of some landscaper. The subject of beauty in concrete has been a hobby with me for a long time. Our company has been under the infliction year after year of building a most hideous, monotonous lot of industrial buildings. I think this condition has arisen because most concrete structures are utilitarian and the majority of the architectural profession is concentrating its thought on the artistic creation of beauty, either in homes, bank buildings or similar structures. **Mr. Barney.**

I think this Institute should organize a committee on Architecture in Concrete. One function of that committee should be to gather pictures of concrete structures and then take up with the American Institute of Architects the question of a more serious consideration of beauty in industrial buildings. These thoughts that Mr. Johnson has left with us today should find some definite form of expression.

R. W. Atwater.—I would like to reiterate the comments Mr. Bates made in his discussion following the symposium on workability, to the effect that we have made a little god out of strength, whereas the real objective is serviceability. **Mr. Atwater.**

If the advancement in the use of concrete, as prophesied by Mr. Johnson, is to take place, concrete must be more uniformly durable. It is constantly becoming more apparent that there are criterions of durability such as maximum density and minimum permeability and shrinkage. Why then should not standardized tests based on these qualities be formulated and made available for the use of engineers and contractors throughout the country, so that the durability of concrete can be controlled by the use of such tests in the same manner that strength is regulated by the use of compression or modulus of rupture tests.

If density, permeability and shrinkage can be used to measure the probable durability of a concrete, and accepted tests for these qualities are made available, then instead of carefully designing a mix for strength and leaving its durability to chance, these tests can be made coincidently with the compression tests and the mix can be designed and regulated for both strength and durability and the resulting material will be a serviceable concrete.

I would like to suggest to the Institute that, working in conjunction with the Bureau of Standards and the American Society for Testing Materials, an effort be made to formulate such tests, even though at first they may not be perfect, and to place them in the hands of engineers and con-

tractors so that concrete can be made durable as well as strong and thus serviceable.

Mr. Powers. E. S. POWERS.—One of my clients was here and heard the talk on concrete buildings and the use of concrete in architecture. He came into my office next morning and wanted to know what I thought about it. He had a good-sized problem that he thought of treating in reinforced concrete. What I have seen of that type of construction here in the East does not encourage me very much in recommending it. There has been some very good work done, but there is a prejudice in favor of old materials that exists in the minds of quite a number of architects. On the other hand, they are alive to the use of cement and are anxious to have a material that will express both strength and beauty in the whole construction. Their ideal, in fact, is to have a system of construction that will express their form without any veneer. What I need is to find out whether there is any system of practice that will guarantee good work in the field when we undertake to build a building in reinforced concrete.

Mr. Johnson. VIRGIL L. JOHNSON.—Speaking as a member of the American Institute of Architects and also as an engineer, I want to compliment Mr. Johnson on his very excellent talk this afternoon. I think he has pictured very clearly why an architect does not want to trust himself to an ordinary concrete structure. When engineers and contractors and concrete mixers can produce a surface which is dependable, I think they will have accomplished a great deal. I do not know why the engineers in this Institute have not pointed to the Walnut Lane Bridge, in Philadelphia, as an example. To my mind, this is a poem in concrete. I think if you will look at it today you will be impressed with the feeling that its engineer has expressed a thought in the design as well as in the surface of the material.

Concrete Primer.

By F. R. McMillan.*

Foreword.

This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete. No claim is made for completeness of detail, and even in presenting the fundamentals, the shortcomings of the method are recognized. The reader is referred to the many excellent texts and the wealth of technical papers and discussions for the answers to questions not found here.

Originally intended only for the beginner in concrete construction, the scope of the primer widened as it grew, in the hope that it would be of use also to the man at the other end of the organization—the man who, though mostly concerned with the success of the project, is least in a position to give attention to the details. It may be that in this latter respect the primer will find its greatest usefulness. Many who have been interested in the cause of better concrete have noted the difficulty of making any real progress until someone in authority has been convinced that good concrete *can* be had, that it *should* be had, and having been so convinced, has sent out the word that it *must* be had. If this series of questions and answers serves in any way to dispel the mystery of concrete control in the minds of those in authority over concrete operations and to leave the conviction that good concrete can be had, and had economically, it will have "made good" even though the man at the mixer may need some further attention.

*Director of Research, Portland Cement Association.

Concrete Primer.

Cement, Mortar, and Concrete.

1. Q. *What is portland cement?*

A. Portland cement is a finely pulverized material consisting of certain definite compounds of lime, alumina, and silica, which when mixed with water has the property of combining slowly with the water to form a hard, solid mass. Technically portland cement is defined in the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials as follows: "Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum."

2. Q. *What are the raw materials used in the manufacture of portland cement?*

A. The two principal materials from which portland cement is made are a calcareous material, such as limestone, chalk, shells, or marl, and an argillaceous material, such as clay, shale, or blast-furnace slag. Very definite proportions of the several components of the raw mixture must be maintained.

3. Q. *How is portland cement made?*

A. The raw materials, which are finely ground and intimately mixed, are heated to the beginning of fusion (about 2,600° F.), usually in great rotary kilns which may be as large as 300 ft. in length by 12 ft. in diameter. The partially fused or sintered material which emerges from these kilns is called "clinker." The clinker is cooled and mixed with a small amount (2 or 3 per cent) of gypsum rock or gypsum plaster. This mixture is then ground to a very fine powder which is the portland cement of commerce.

4. Q. *What causes the hardening of portland cement?*

A. When portland cement is mixed with enough water to form a paste, the compounds of the cement react with the water to form both crystalline and jelly-like products. These products adhere to the aggregate and to each other and become very hard. If the materials are kept moist, the reactions may continue for years, and thus the product continues to become stronger over a long period of time.

5. Q. *What is meant by "setting of cement"?*

A. When mixed with water, cement forms a paste which remains plastic for a short time, but as the reactions with the water proceed, the mix begins to stiffen or "set." At this stage of the "setting" it is still possible to disturb the material and remix without injury, but as the reactions between the cement and water continue, the mass loses completely its plasticity, and if disturbed or remixed, the strength will be seriously impaired. This early period in the hardening of cement is spoken of as the "setting period," although there is no well-defined break in the hardening process. Once the mass has definitely hardened, the chemical action continues, building up a firm internal structure which gains in hardness and strength as it proceeds. The "set" and subsequent hardening process are the same whether the cement is used alone or in combination with aggregates.

6. Q. *What is meant by "initial set" and "final set" in reference to cement?*

A. Among the other tests for cement, there is a test for "time of setting" in which the initial and the final set are determined by the application of the Vicat or Gillmore needles to the smooth surface of a pat of cement paste. The application of both of these methods consists in determining the time when the paste has stiffened to a definite degree as indicated by the failure of the needles to penetrate a prescribed amount or to make appreciable indentations on the surface. The weights and areas of the needles are purely arbitrary and the conditions of hardness which identify the periods selected for the initial and final set are only passing stages of the hardening process, and are not points that mark definite changes in the manner of behavior of the cement.

7. Q. *What is concrete?*

A. Concrete is a mixture in which a paste of portland cement and water binds fine and coarse materials, known as "aggregates," into a rock-like mass as the paste hardens through the chemical action of the cement and the water.

8. Q. *What is mortar?*

A. Mortar is a mixture similar to concrete in which no large-size aggregates are used.

9. Q. *What are aggregates?*

A. Aggregates are the inert materials used in concrete and mortar mixtures principally to increase the mass and reduce the cost of the product. Aggregates are usually divided for convenience into fine and coarse. Sand is the most common form of fine aggregate, and gravel or crushed rock, the most common form of coarse aggregate.

10. Q. *Is portland cement a modern product?*

A. Portland cement was patented by Joseph Aspdin in 1824. At first its use was confined to mortars for sub-aqueous masonry, but with the perfection of its manufacture, it came into general use for masonry mortars and in concrete as a substitute for stone masonry. Following the invention of reinforced concrete, it became one of the principal manufactured products of commerce. In the last quarter century the use of portland cement has continued to expand until now almost no engineering or construction project is carried out without portland-cement concrete in some part of the work.

11. Q. *Has the practice of concrete construction been developed solely by experience, or has scientific research aided in its growth?*

A. Concrete and reinforced concrete have had the benefit of more engineering study and research than any other building material. In addition, because of its great adaptability and usefulness, concrete construction has developed through experience as probably no other engineering art.

12. Q. *Are the properties of concrete well enough known to enable safe and durable structures to be built?*

A. The principles of concrete making and the laws of concrete behavior are well enough established through experience and research to make possible the safe design of structures to meet the requirements of any engineering project.

13. Q. *Can these principles and laws of concrete construction be explained in simple terms?*

A. Yes, but their complete mastery will require thoughtful study.

14. Q. *What are the essential requirements for a successful concrete structure?*

A. The concrete must have sufficient strength to carry the loads imposed. The concrete must be able to endure under the conditions of exposure to which it will be subjected. The concrete must be economically produced in comparison with other materials equally strong and durable which might be used. Thus the requirements may be briefly stated as: *strength, durability, economy.* In this primer, it is proposed to develop in logical sequence the many considerations necessary to produce concrete structures in which these requirements will be in proper balance.

Factors Affecting the Strength of Concrete.

15. Q. *What is the most common measure by which the quality of concrete is judged?*

A. Compressive strength.

16. Q. *Does a high compressive strength indicate that the concrete possesses other desirable qualities in a high degree?*
A. It does to a very considerable extent, though not always. The factors which improve the compressive strength generally improve the other qualities.

17. Q. *When is a high compressive strength not a sufficient guarantee of the quality of a structure?*
A. When it is not accompanied by watertightness, high compressive strength may lead to a false security in the quality of the structure.

18. Q. *What are the factors governing the compressive strength of concrete?*
A. The principal factors governing the compressive strength of concrete are the following: *curing conditions, age, characteristics of the cement, quantity of the cement, quantity of mixing water, influence of the aggregates, time of mixing, and conditions of test.*

CURING OF CONCRETE:

19. Q. *What conditions are required for the continued hardening or increase in strength of portland cement concrete?*
A. The continued chemical reaction upon which the increase in strength depends requires the presence of water and favorable temperatures.

20. Q. *What is meant by curing?*
A. The term "curing" is used in reference to the continuing of these chemical reactions. In the presence of water and favorable temperatures, curing can proceed. It is through this curing that the internal structure of the concrete is built up to provide strength and watertightness.

21. Q. *What is meant by properly cured concrete?*
A. This term is more or less indefinitely used to indicate that the chemical reaction has progressed to a point that insures satisfactory performance of the concrete in the structure.

22. Q. *Is there any measure of the completeness or adequacy of curing?*
A. No, except with reference to the strength or some other specific property of the concrete, such as permeability.

23. Q. *Will concrete continue to harden or increase in strength if there is no water present?*
A. No. With a total absence of water, concrete does not continue to harden or increase in strength.

24. Q. *Can concrete, which has dried out at the early ages, be restored by moist curing?*
A. Yes, but at some sacrifice of strength. The most favorable period for curing is that during the first few days or weeks. Con-

tinued moist curing after a period of drying out will continue to add strength, but at a less rapid rate.

25. Q. *Does additional curing improve the quality of concrete in other ways than in strength?*

A. Yes. All desirable properties of the concrete—wear resistance, bond strength, watertightness, etc.—are improved by additional curing.

26. Q. *How much water is required for the hardening of concrete?*

A. This cannot be answered in definite figures. As the hardening continues, water continues to enter into combination with the cement, forming a part of the permanent internal structure. The proportion of water actually entering into the combination may vary from as little as 5 to as much as 25 or 30 per cent of the cement by weight, depending upon the age and curing conditions.

27. Q. *Does the amount of water ordinarily used with cement in mortar and concrete greatly exceed that required for completion of the chemical reactions?*

A. Yes. In order to place concrete properly, fluidity or plasticity is required far in excess of that which would be furnished by an amount of water only sufficient to complete the reactions.

28. Q. *What part has temperature in curing?*

A. The reactions proceed more rapidly, the more favorable the temperature.

29. Q. *What temperatures are unfavorable for curing?*

A. Temperatures below 50°F. are considered unfavorable for the early curing period. Below 40°F., the curing is greatly retarded and at freezing temperatures, it is extremely unreliable. Fig. 1 shows the effect of temperature on the strength of concrete cured in a moist atmosphere and tested wet.

30. Q. *What is the effect of freezing on concrete?*

A. Freezing of fresh concrete is very harmful. It disrupts the mass and permanently impairs the strength.

31. Q. *Can concrete, which has been frozen, be recovered?*

A. If thawed out without being disturbed and good curing conditions are maintained, concrete will acquire considerable strength, but available tests indicate that it never will attain its full potential value.

32. Q. *What special precautions should be taken in case concrete has frozen?*

A. It is particularly important to keep it from drying out after having thawed. Flooding with warm water and keeping the surrounding air at temperatures above 70°F. would be the best treatment. Because of its low absolute humidity, winter air

has a great avidity for water when its temperature is raised, so that an unprotected concrete structure dries out very rapidly in such air.

AGE:

33. Q. *What part does the age play in the strength of concrete?*

A. As pointed out in the answers to previous questions, concrete continues to gain in strength for a long period, provided the temperature and moisture conditions are favorable. Thus, the

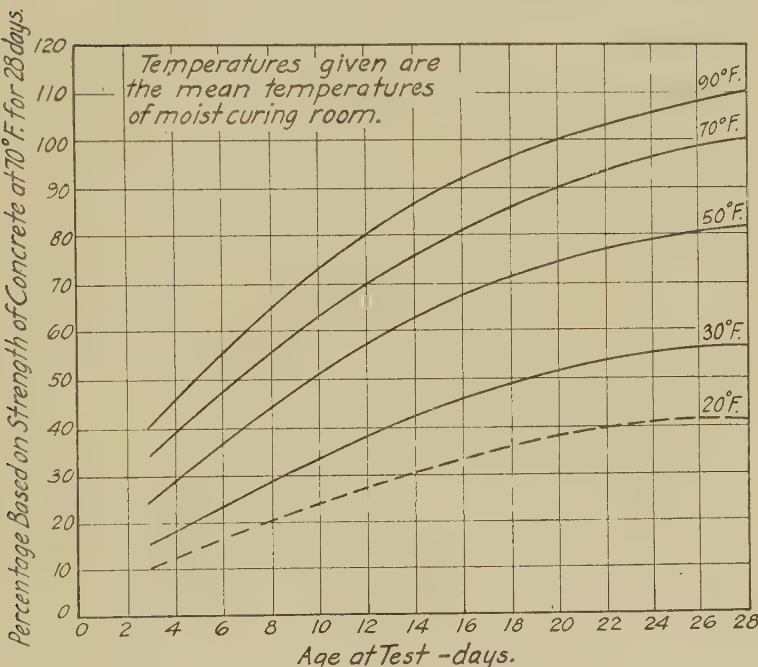


FIG. 1.—EFFECT OF TEMPERATURE DURING CURING ON THE COMPRESSIVE STRENGTH OF CONCRETE. DATA TAKEN FROM BULLETIN 81, ENGINEERING EXPERIMENT STATION, UNIVERSITY OF ILLINOIS.

strength is necessarily a function of the age. Fig. 2 shows the age-strength relation for several grades of concrete made from a mixture of four brands of cement and average sand and gravel aggregate.

34. Q. *At what age is concrete generally tested to ascertain its quality and to compare with standard specifications?*

A. Generally at 28 days. Oftentimes, it is convenient to know something of the quality of the concrete in advance of this period and 7-day and earlier tests are now quite commonly made.

CHARACTERISTICS OF THE CEMENT:

35. Q. *How do the characteristics of the cement affect the compressive strength of concrete?*

A. All portland cements behave similarly, although the gain in strength with age is not always the same. Some cements gain their strength more rapidly at first, while others show greater

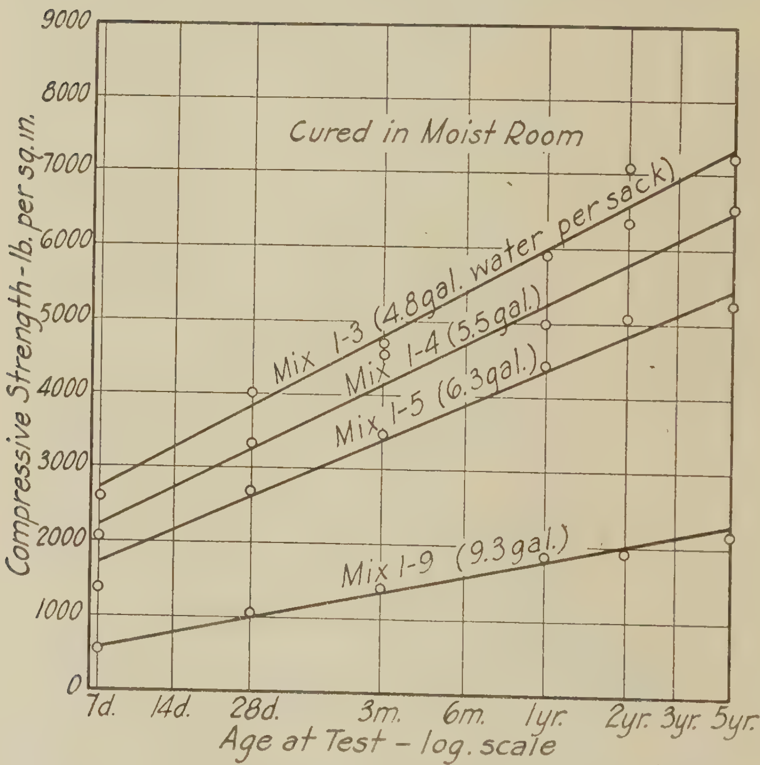


FIG. 2.—INCREASE OF STRENGTH OF CONCRETE WITH AGE. MOIST CURING, SPECIMENS TESTED WET.

increase for the longer periods. Therefore, the compressive strength of concrete at any age is affected by the characteristics of the cement.

36. Q. *Is there a minimum requirement as to strength for portland cement at given ages?*

A. Yes. There is a standard specification which is practically universal in the United States. This is known as the "Standard Specifications and Tests for Portland Cement" (Serial Desig-

nation: C 9-26) of the American Society for Testing Materials. To pass these specifications, the cement must show among other requirements a certain tensile strength at 7 and 28 days when tested in the prescribed manner.

37. Q. *Is the value of a cement as a construction material measured solely by its 7 and 28-day strengths?*

A. The value of a cement to a construction project must be measured by its long-time performance, and not by its early strength. The specifications recognize this fact in that the limits set for the early strength may be considered as the minimum desirable values.

38. Q. *Are the standard specifications a sufficient guarantee that the cement will be satisfactory for concrete construction?*

A. Yes. The standard specifications have been built up through years of experience and experimentation to protect the buyer of cement on all elements of quality. The fact that different cements exceed the minimum early strength requirements by different amounts is not of great significance; such differences as do exist become less and less with age.

39. Q. *How does the quantity of cement influence the strength of concrete?*

A. Just as with other materials, the more that is used, the greater the strength. The more cement that is used with a given quantity of aggregates the less the water needed to produce the desired consistency, and therefore, as explained in the next question, the greater the strength.

QUANTITY OF MIXING WATER:

40. Q. *How does the quantity of mixing water affect the strength?*

A. The strength of a hardened cement paste is found to vary materially with the amount of water used in mixing. The use of about 2¾ gal. water for each sack of cement makes a paste that can be stiffly molded. Water in excess of this amount renders the mix more plastic, but has the effect of diluting the paste and reducing the potential strength.

41. Q. *Is there any fixed relation between the quantity of water used in the paste and the strength?*

A. Yes. Tests have shown a very constant relation for given materials and standardized conditions of making, curing, and testing.

42. Q. *Does this relation between the strength and water content of the paste hold good when aggregates are used in the mix?*

A. Yes, provided the quantity used is not so great as to make the mixture too stiff to mold properly, and that clean aggregates of structurally sound particles are used. Fig. 3 shows the

results of Abrams' tests on a large number of mixes. This figure shows the 28-day strength for a given set of conditions to be represented by a fairly definite curve.

43. Q. *Have other tests shown the same relation between strength and quantity of mixing water as that first found by Abrams?*

A. Yes, the general character of this water-cement ratio-strength relation for plastic concretes has been clearly established by many subsequent tests by Abrams and others. The curves all

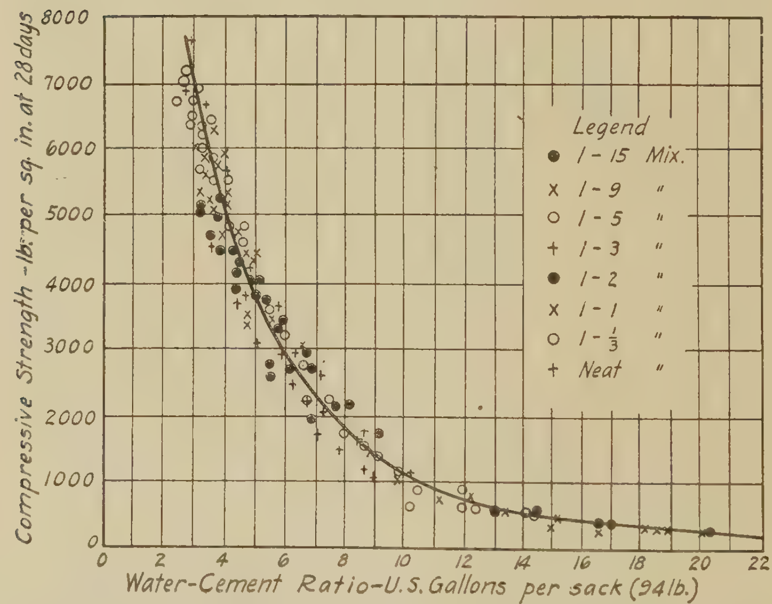


FIG. 3.—RELATION BETWEEN STRENGTH OF CONCRETE AND THEIR WATER CONTENT. DATA FROM BULLETIN 1, STRUCTURAL MATERIALS RESEARCH LABORATORY, CHICAGO, ILL.

have the same characteristics though the strengths for the same water-cement ratio are not exactly the same.

44. Q. *What factors cause the difference in strength for given water-cement ratios and standardized conditions of making and testing?*

A. Principally differences in materials. As pointed out in the answer to Question 35, not all cements have the same strength for a given set of conditions. Likewise, as pointed out in answer to Question 45, differences in aggregates have some effect.

In a series of questions and answers to follow (Questions 86 to

100) the use of the water-cement ratio-strength relation in designing concrete mixtures is explained.

INFLUENCE OF THE AGGREGATES:

45. Q. *How does the character of the aggregates influence the strength of concrete?*

A. The principal factors are the capacity to absorb water, the surface characteristics, the shape and size of the particles, and the combination of sizes, that is, grading.

46. Q. *How does the absorption of the aggregate affect the strength?*

A. By extracting water from the mix, it has the effect of reducing the quantity of mixing water, and therefore, increasing the strength.

47. Q. *How do the surface characteristics affect strength?*

A. By affecting the adhesion of the cement paste to the aggregate particles. The presence or absence of dust, the roughness and texture affect the adhesion.

48. Q. *How do the shape and size of the particles affect the strength?*

A. By affecting the grip or bond with the cement paste and by affecting the amount of paste that is required for a given mix. Also, by introducing planes of weakness where large unbroken areas of the aggregate surface occur.

49. Q. *Does the grading of the aggregates affect the strength of concrete?*

A. When the water-cement ratio is the same and the mixes are plastic, considerable changes in grading affect the strength of the concrete only to a small degree. The principal effect of changing the aggregate grading is to change the amount of cement and water needed to make a plastic mixture. (See Question 40, also 108 to 116.)

50. Q. *Does the strength of the aggregate particles affect the strength of concrete?*

A. Except for very weak aggregates, the strength of the particles affects the strength only slightly. Strength of particles in excess of the potential strength of the concrete itself is of no avail. Weak aggregate particles, of course, limit the potential strength of the concrete.

51. Q. *For a given water-cement ratio and standardized conditions of test, are the differences in strength of concretes made from different aggregates important?*

A. They may be, but generally, if allowance is made for reduction in the water-cement ratio by the absorption, the ordinary sizes of the more common aggregates will give about the same results.

52. Q. *What is the most important requirement of a concrete aggregate?*

A. That it shall not change in structure or composition under the condition to which it will be exposed.

TIME OF MIXING:

53. Q. *How does the time of mixing affect the strength of concrete?*

A. Increased time of mixing increases the strength. The increase in strength is rapid for increase in time up to about 1 minute; beyond this, the increase is much less rapid. Beyond 2 minutes, there is little strength to be gained by further mixing.

54. Q. *What time of mixing is recommended?*

A. For most work at least one minute is desirable. The time of mixing should refer only to the actual mixing period after all materials are in the mixer. Where watertight concrete is required, or under severe conditions of exposure, 1½ or 2 min. should be the minimum.

CONDITIONS OF TEST:

55. Q. *What are the conditions of test which affect the strength of concrete?*

A. The principal factors affecting the strength results are:
Method of making specimen.
Size and shape of the test specimen.
Irregularities in the test specimen.
Moisture content of the test specimen.
Method of capping before testing.
Type of bearing block in testing machine.
Rate of application of the load.

56. Q. *How does the method of molding affect the test results?*

A. Unless the specimens are carefully molded, erratic and irregular results will be obtained. The concrete should be placed in the mold in several layers, rodding each layer before more concrete is introduced. The mold should be held firmly in a vertical position during filling and stand on a base plate that is a true plane. (See A. S. T. M. Standard Methods of Making Compression Tests of Concrete—Serial Designation: C 39-27.)

57. Q. *What size and shape of specimen are recommended?*

A. Cylinders in which the height is two times the diameter, and the diameter at least four times the size of the largest aggregate particles.

58. Q. *What irregularities are to be avoided in the test specimen?*

A. Uneven ends, specially convex ends, ends not parallel, axes not vertical. A convexity of 0.01 in. on the end reduces the strength about 25 per cent. Ends out of parallel or axes not vertical by more than ¼ in. will reduce the strength even though a spherical bearing block is used.

59. Q. *Does a wet specimen test higher than a dry one?*

A. No. Drying out before test will increase the strength from 20 to 30 per cent.

60. Q. *What methods of capping are recommended?*

A. Whenever possible, caps should be placed at time of making according to the A. S. T. M. Standards. Otherwise the specimen should be capped or bedded with a thin layer of gypsum plaster about an hour before testing.

61. Q. *In testing cylinders, why is it desirable to use a spherical bearing block?*

A. A spherical bearing block is necessary in order to insure the uniform distribution of the load over the cross-section of the specimen.

62. Q. *How does the rate of application of the load affect the results of compression tests?*

A. The faster the load is applied, the greater is the indicated strength. A. S. T. M. "Standard Methods of Making Compression Tests of Concrete" (Serial Designation: C 39-27) require that the load be applied uniformly and without shock, and at a rate such that the moving head of the testing machine travels about 0.05 in. per minute when the machine is running idle. This rate applies to the test of the standard 6 x 12-in. cylinder.

63. Q. *Are there standard methods of making and testing concrete specimens?*

A. Yes. A. S. T. M. Standard Method of Making and Storing Specimens of Concrete in the Field (Serial Designation: C 31-27). A. S. T. M. Standard Methods of Making Compression Tests of Concrete (Serial Designation: C 39-27).

These should always be followed where possible. In any case, great care should be used in all operations of making and testing.

Problems of Proportioning.

64. Q. *What proportions of aggregate and cement are required for concrete?*

A. That depends on the requirements of the work.

65. Q. *What are the factors governing the proportions of cement and aggregates?*

A. The principal factors are:

The requirements as to placing,

The strength needed,

The quality of the concrete necessary to meet the given conditions of exposure,

Considerations of economy.

CONSISTENCY AND WORKABILITY:

66. Q. *How do the requirements of placing affect the proportions of aggregates and cement?*

A. The size and shape of the structural member and the amount and disposition of the reinforcement fix definite limits to the consistency and workability of the concrete. Both the consistency and workability are dependent upon the relative quantities of aggregates and cement.

67. Q. *What is meant by each of the terms "consistency," "plastic," and "workability" as applied to concrete mixtures?*

A. "Consistency" is a general term relating to the character of the mix with respect to its state of fluidity. Consistency embraces the entire range of fluidity from the driest to the wettest possible mixtures and requires a qualifying term for definiteness. Thus, we say that concrete has the consistency of damp earth or of thick mush.

The term "plastic" is used to describe a consistency of freshly mixed concrete which can be readily molded, but which changes form slowly when the mold is removed. A plastic consistency is between the dry, crumbly consistencies on the one hand, and the very fluid or watery consistencies on the other. A mass that is "plastic" does not crumble. It flows sluggishly and without segregation.

The term "workability" is used with reference to concrete mixtures to describe the ease or difficulty which may be encountered in placing the concrete in a particular location. It involves not only the thought of a consistency of concrete, but also the condition under which it is to be placed—size and shape of the member, spacing of reinforcement rods, or other details interfering with the ready filling of the forms. A stiff plastic mixture with large aggregate, which is workable in a large open form, would not be workable, for example, in a thin wall of complicated reinforcing details.

68. Q. *What is the quality of a plastic consistency that makes this consistency desirable for placing concrete?*

A. In a plastic concrete, the aggregate particles are virtually floated in the cement paste. This insures the complete incorporation of all the particles and eliminates the possibility of voids between them.

69. Q. *If the cement paste is not sufficient in amount to float all of the aggregate particles, what is the condition?*

A. The mass is granular or crumbly. It is not mobile and cannot be molded in the forms except by heavy ramming, and even then may be honeycombed or have unfilled voids.

70. Q. *If the paste is ample in amount, but is too thin and watery to float the aggregate particles, what is the result?*

A. The mixture will segregate, the thin paste separating from the coarse aggregate particles to leave a non-homogeneous and porous concrete.

71. Q. *Are the dry, crumbly consistencies, or the thin, watery consistencies desirable in concrete?*

A. No. For uniform, watertight, and weather-resistant concrete, it is essential that the concrete be placed in a plastic consistency. Tests and experience have clearly shown that neither the over-wet mixes, nor the dry, crumbly ones (except when placed and rammed with greatest care) give concrete free from voids, sand streaks, or stone pockets.

72. Q. *In a mass of freshly-mixed concrete of plastic consistency, is all of the space between the aggregate particles filled with the cement paste?*

A. Yes, except for a very small amount (less than 1 per cent usually) of air which may be entrained in the paste.

73. Q. *What happens to an already plastic concrete if still more paste is added?*

A. The aggregate particles are pushed apart and the volume of concrete is increased by exactly the volume of the paste added.

74. Q. *With this additional paste, is the mass more fluid or workable?*

A. Yes. Adding paste makes the concrete more workable; adding aggregates makes it stiffer or less workable.

75. Q. *What happens to an already plastic concrete if water alone is added?*

A. The added water increases the volume of the paste and at the same time makes it more fluid. Thus, the concrete itself is increased in volume by the volume of water added and is likewise made more fluid.

76. Q. *What happens if only cement is added?*

A. The volume of paste is increased; likewise, the concrete volume. In this case, the paste itself is stiffened, but the fluidity of the concrete may be somewhat increased due to the larger volume of paste even though the paste itself is somewhat stiffer.

TESTS FOR WORKABILITY AND CONSISTENCY:

77. Q. *Is there any measure of consistency or workability?*

A. No absolute measure has yet been devised of either consistency or workability.

78. Q. *Is the slump test of value as a measure of consistency or workability?*

A. The slump test can be very useful as an indication of consistency and with certain mixes also of workability.

79. Q. *How is the slump test made?*

A. A mold in the form of a frustum of a cone, 12 in. high, base diameter 8 in., top diameter 4 in., is filled with the freshly-mixed concrete in 3 layers; each layer being rodded with a ⅝-in. bullet-pointed rod 25 times. When filled, the top is struck off and the mold lifted. The amount which the mass settles as the mold is removed is termed the "slump." A small slump indicates a stiff consistency and a very large

FIG. 4.—THE SLUMP TEST. LEFT, FILLING THE SLUMP CONE. RIGHT, MEASURING THE SLUMP.

slump, a very wet consistency. Fig. 4 shows the filling of the slump cone and measuring the slump.

80. Q. *Why is the slump test not an absolute measure of consistency or workability?*

A. Because it does not distinguish between mixes of a different character. For example, a harsh coarse mix cannot be said to have exactly the same consistency as one with a large proportion of sand even though they have the same slump. In the case of workability, it is even more difficult to judge the character of the mix by the slump.

81. Q. *When is the slump test a good guide to consistency?*

A. When the mixes and the aggregates remain unchanged, the slump test gives a good measure of any changes in consistency. Where the materials are being accurately measured, a change in the slump as the work proceeds indicates either a change in the grading of the materials, or a change in water content. The slump test under these conditions can be a good basis for control.

82. Q. *When does the slump test provide a useful indication of workability?*

A. If mixes have been specified which are truly workable for the given structure, then small changes in slump may be considered as indicating corresponding changes in workability.

STRENGTH OF CONCRETE TO USE:

83. Q. *What strength of concrete is needed for different classes of work?*

A. No fixed values can be given for any class of work. Large members of low-strength concrete may occasionally prove more desirable than small members of high strength. Ordinarily, that strength is most desirable which gives the greatest economy in design, but frequently other considerations such as clearances, head room, and weight of structure may be of greater concern than the mere question of cost of the concrete itself. Even where this cost is the only consideration, variations in cost of constituent materials and in plant and labor charges make it undesirable to attempt a too arbitrary classification of values.

84. Q. *Has it not been the practice to adopt certain arbitrary strengths as suitable for different classes of work?*

A. Yes, quite generally, and these values can usually be taken as a guide. It is, however, becoming more the practice to specify the strength of concrete for a given set of conditions just as different grades of steel are specified for different classes of structures.

85. Q. *How can the strength of concrete be predetermined?*

A. The advance in knowledge of concrete mixtures in recent years makes it possible to "design mixtures" for given conditions with considerable accuracy much the same as the science of metallography has made it possible to produce different steels for special purposes.

DESIGN OF CONCRETE MIXTURES FOR STRENGTH:

86. Q. *What is the basis of the design of concrete mixtures for strength?*

A. The relation between the quantity of mixing water and the strength of the concrete gives a good basis for the design of mixtures.

87. Q. *Is the Abrams' curve shown in Fig. 3 a good basis for design?*

A. In the absence of reliable tests on the materials to be used, this curve makes a satisfactory basis for design for normal temperatures and where the job conditions, particularly with reference to the water-cement ratio, are to be carefully controlled. A somewhat lower set of values should be used for the job where accurate measurement of the water is not possible.

88. Q. *What is the most desirable way of utilizing the water-cement ratio principles in the design of concrete mixes?*

A. To have a reliable series of tests made from the materials to be used covering a series of water-cement ratios so that a strength curve for the particular conditions can be established. With this water-ratio strength curve established mixes can be designed as required for any strength.

89. Q. *How many tests should be made and what precautions followed to obtain a reliable curve for a given job?*

A. That would depend upon the importance of the work and the previous experience with the given materials. A minimum would be tests for five different water ratios with three specimens for each condition of test. The mixes should be of approximately the consistency that will be required. All specimens and tests should be made by experienced or properly qualified persons and following the standards of the American Society for Testing Materials.

90. Q. *Should the specimens be cured on the job, or under the standardized method for laboratory specimens?*

A. They should preferably be cured in a moist atmosphere at approximately 70° F. In any case, the temperature should be constant over the curing period. Otherwise, there would be no proper basis for comparing the results with known standards or of duplicating them later on the work, if changes in materials or other conditions make further tests necessary.

91. Q. *Would it not be desirable to have some specimens cured on the work under the prevailing conditions?*

A. Yes. Such tests are desirable both as a check on the quality of concrete going into the structure and as a measure of the severity of the exposure. These tests, however, will be of greater value when supplemented by the tests under standardized conditions; otherwise, it would not be possible to provide correctly for changes in job conditions.

92. Q. *Is it desirable that regular tests be carried out during the progress of the work?*

A. Yes. Particularly so where widely varying temperature conditions are to be encountered.

93. Q. *Is there any way of anticipating the water-cement ratio required for different strengths for temperatures other than 70°F. under which Abrams' tests were carried out?*

A. Yes. Fig. 5 will serve as a useful guide. The curves in this figure are based on the data given in Bulletin 81 of the University of Illinois Experiment Station, covering tests on concrete cured in a moist atmosphere at various temperatures. The reduction coefficients from these tests (see Fig. 1), have been applied to the Abrams' curve Fig. 3 to give the values shown.

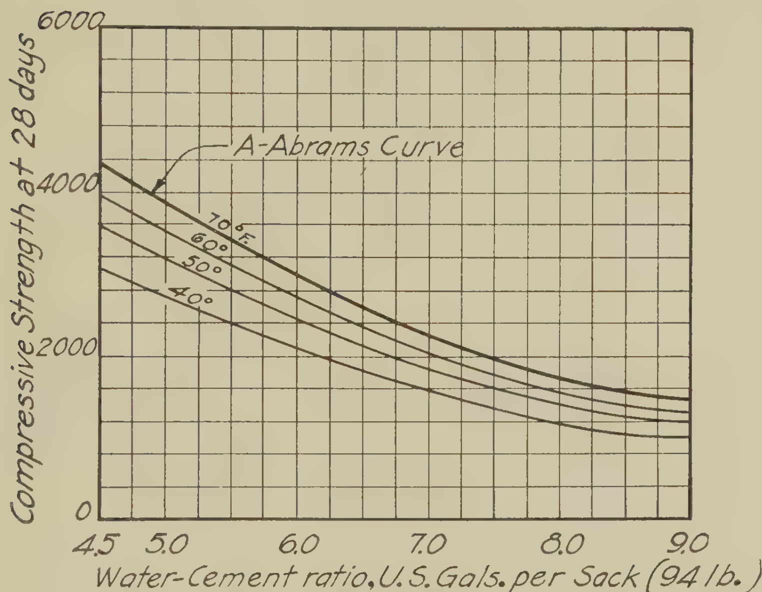


FIG. 5.—WATER-CEMENT RATIO-STRENGTH RELATION FOR MOIST-CURED CONCRETE AT DIFFERENT TEMPERATURES. CURVE A FOR 70° F. THE SAME AS CURVE IN FIG. 3. CURVE FOR OTHER TEMPERATURES DEDUCTED FROM CURVE A BY THE DATA FROM FIG. 1.

94. Q. *What is the procedure for designing mixtures by the water-cement ratio method to give a desired strength of concrete?*

A. The water-cement ratio to be used for the desired strength of concrete is selected either from the curves of Fig. 3 or 5, or on the basis of tests made for the particular job as described in the answers to Questions 89 and 90. Having selected the water-cement ratio for the strength desired, trial batches can be made up in which cement and water are mixed in this proportion and aggregates added until a workability suitable for the structure is obtained.

95. Q. *Is it necessary to take into account the surface or free water carried by the aggregates?*

A. Yes, the water-cement ratio, which fixes the strength of concrete, is based on the quantity of water actually entering the mix. If some of it is introduced into the mix in connection with the aggregate, the amount to be added in the usual way at the time of mixing must be reduced accordingly.

96. Q. *If dry aggregates are used, some water is absorbed from the mix during the operation of mixing and placing. Should this be taken into account?*

A. Yes. If the amount absorbed during this period is in measurable quantities, allowance can be made by increasing accordingly the amount added at the mixer.

97. Q. *Should allowance be made for any water that is lost by evaporation during transporting and placing?*

A. Yes. The effective water-ratio which determines the strength can be considered the net amount remaining in the cement paste after the concrete is in place and thoroughly consolidated. Water extracted from the mix, after the concrete is in place and while still plastic, in a manner which permits of the consolidation of the mass, may be deducted in computing the effective water-cement ratio. This extraction of water is highly beneficial so long as the concrete continues to consolidate without losing its plasticity. The extraction of water beyond the point where its place is not taken by further consolidation of the mass is undesirable as it leaves pores or voids which will hasten the drying out of the concrete.

98. Q. *How can the surface or free water carried by the aggregates be determined?*

A. There are several methods which can be used. A description of some of these appears at the end of this primer.

99. Q. *Can the surface water carried by the aggregates be estimated for average conditions?*

A. Only in the roughest sort of way. The following values can be taken in the absence of any tests, but it should be remembered that these are subject to considerable variation:

APPROXIMATE AMOUNTS OF FREE WATER IN AVERAGE AGGREGATE.

Condition of Aggregate	U. S. Gallons per Cubic Foot
Very wet sand	¾ to 1
Moderately wet sand	½
Moist sand	¼
Moist gravel or crushed stone	¼

AGGREGATE PROPORTIONS:

100. Q. *What is the procedure for determining the proper quantities of aggregates to be added?*

A. A good way is to begin with a mix that previous experience indicates should approximate the requirements for the particular job. This mix can be made up in the usual way except that the water should be measured to give the water-cement ratio selected (correcting, of course, for surface water carried by the aggregates, or for losses through absorption as the case may be). From the appearance of this first mix, it will be easy to decide upon the quantities to use for the second trial batch. After a few trials the most desirable proportions can be selected.

In these trial batches, a record should be kept of the amount of cement, fine and coarse aggregate used, and of the quantity of concrete produced in each case. From this record for several batches, it will be possible to compare the different mixes on the basis of cost of materials as well as for workability.

101. Q. *Does the amount of aggregate added, or the proportion of fine to coarse affect the strength of the concrete?*

A. So long as the mass remains plastic and workable, varying these quantities within the limits ordinarily desired will make no appreciable change in strength for a given water-cement ratio.

102. Q. *Does the grading of the aggregate affect the strength of the concrete where the water-cement ratio is fixed?*

A. So long as the mixes remain plastic and workable, ordinary variations in grading will make no appreciable changes in strength for a given water-cement ratio.

103. Q. *Under the old method of specifying concrete on the basis of arbitrary proportions, such as 1:2:4, how does the grading of aggregate affect the strength?*

A. Where fixed proportions are specified, differences in grading require different amounts of mixing water to produce the desired workability, and therefore, give different strengths.

104. Q. *Where mixes are to be designed for a given water-cement ratio, is the grading of the aggregate an important consideration?*

A. Grading is important in that it affects materially the quantity of aggregate which can be used with a sack of cement and its fixed proportion of water to obtain a desired consistency.

105. Q. *What should be the guide in fixing the proportions of cement, and fine and coarse aggregate?*

A. The ease and convenience in placing and the cost.

106. Q. *Are the mixes which place most easily likely to be the lowest in cost when designing mixes for a given water-cement ratio?*

A. No. On the contrary, these are two opposing factors. For a given water-cement ratio, the principal item in the cost of concrete is the amount of aggregate that can be used with a sack of cement and its fixed proportion of water—the greater the amount of aggregate, the lower the cost. Ease of placing, on the other hand, results from using a small amount of aggregate which gives greater fluidity. Thus, the tendency when designing mixes for a fixed water-cement ratio should be to use the largest amount of aggregate possible without sacrificing proper workability or increasing unduly the cost of placing.

107. Q. *What is meant by the term "yield" and "cement factor"?*

A. "Yield" refers to the quantity of concrete produced from a sack of cement.

"Cement factor" refers to the quantity of cement in a cubic yard of concrete. Thus, if 5 sacks of cement are required for one cubic yard of concrete, the cement factor is 1.25 bbl. per cu. yd. and the yield is 5.4 cu. ft. of concrete per sack of cement.

Sometimes, the term "yield" is used to indicate the volume of concrete produced in terms of the quantity of combined aggregates used. Thus, a mixture of 4 cu. ft. of combined aggregates with 1 bag of cement giving 4.4 cu. ft. of concrete would have a yield of 1.10.

108. Q. *Does the relative proportion of fine and coarse affect the yield, that is, amount of aggregate that can be used with a sack of cement and a fixed proportion of water?*

A. Yes, very materially, as will be explained in the answers to the next few questions.

109. Q. *Does a high proportion of coarse or a high proportion of fine aggregate tend to increase the yield where there is a fixed ratio of water to cement?*

A. Generally, a high proportion of coarse aggregate (within the limits given in answer to Question 116) permits the use of a greater quantity of combined aggregate for a given consistency.

110. Q. *How does the yield of an all-sand mixture compare with that where sand and coarse aggregate are used in proper proportions?*

A. Where the quantity of water is limited to a definite proportion of the cement, the yield of an all-sand mixture is relatively low. See also answer to Question 112.

111. Q. Why does an all-sand mix or one with a very high proportion of fine aggregate reduce the yield when used with a fixed water-cement ratio?

A. Because of the large surface areas to be coated with the paste, the excessive quantity of fines stiffens the mix rapidly so that only a small yield can be obtained and the mix remain plastic.

112. Q. What is the relative effect of additions of fine and coarse aggregate?

A. The following table shows how the addition of fine material stiffens the mix of fixed water-cement ratio more than corresponding additions of coarse. These data are from actual tests using sand and gravel aggregates, the mixes indicated being on the basis of dry, compact volumes:

Mix Dry, Compact Volumes	Water Cement Ratio U. S. Gallons per Sack	Yield Cubic Feet Concrete from 1 Sack Cement	Slump Inches
1 : 1½ : 3	5¾	4.2	8.0
1 : 2½ : 3	5¾	5.0	0.5
1 : 1½ : 4	5¾	4.9	4.5
1 : 3 : 0	6	3.4	0.8
1 : 2 : 3	6½	4.7	7.8
1 : 3 : 3	6½	5.7	0.6
1 : 2 : 4	6½	5.4	6.0

The table shows that adding 1 cu. ft. of sand to the 1 : 1½ : 3 mix to obtain the 1 : 2½ : 3 mix reduces the slump from 9.3 to 0.5 in., while adding 1 cu. ft. of coarse aggregate to obtain the 1 : 1½ : 4 reduces the slump only to 4.5 in. Similarly, in the second group of mixes for a water-cement ratio of 6½ gal. per sack, the extra cubic foot of sand has a much greater effect than does the extra cubic foot of coarse aggregate. The all-sand mix (1 : 3 : 0) in the first group shows a very stiff consistency and a very low yield as pointed out in answer to Question 110.

113. Q. Can the variations in cement factor, due to variations in proportions of fine and coarse aggregate for concrete of constant slump and water-cement ratio be illustrated by an example?

A. The tests from which the above table was taken show the following mixes and cement factors for a slump of 5 to 6 in. and a water-cement ratio of 5½ gal. per sack:

Mix Dry, Compact Volumes	Ratio, Coarse to Fine	Cement Factor Barrels per Cubic Yard
1 : 1 : 4	4.0	1.50
1 : 1½ : 3½	2.3	1.47
1 : 2 : 3	1.5	1.45
1 : 2½ : 1	0.4	1.85

Note how the cement factor is greatly increased by the mix with the very high sand ratio, 1 : 2½ : 1.

114. Q. *Is the comparison shown in the answer to Question 113 typical?*

A. For average materials, a condition similar to that shown will usually be found. For mixes with proportions of fine and coarse aggregates about right for easy placing, a lower cement factor is found than for those mixes with either a larger or smaller proportion of fine to coarse.

115. Q. *In view of the facts brought out in the answers to Questions 105 to 114, is it a safe practice in designing mixes for a given water-cement ratio, to adjust the proportions of fine and coarse aggregate solely on the basis of ease of placing and economy?*

A. Provided the limitations given in the answer to Question 116 are observed, aggregate proportions can be selected safely on the basis of best total economy, considering cost of handling and placing as well as materials.

116. Q. *What are the limitations in proportions of aggregate to cement and in the ratio of fine and coarse aggregate that should be observed in designing concrete mixes for a given water-cement ratio?*

A. The total quantity of aggregate to be used with a sack of cement and its fixed proportion of water should be such as to avoid both overwet and extremely dry mixes. The proportions of fine and coarse should be such as to avoid foolish extremes in either direction. Even where it gives the lower cost, too high a ratio of fine to coarse is undesirable as it results in concrete of a lower weight and greater expansion and contraction with changes in moisture content. Too high a ratio of coarse to fine aggregate is undesirable as it produces a harshness of the mix that makes placing difficult and tends to the production of honeycomb and stone pockets.

A desirable range in the proportions of fine to coarse for average materials is indicated by the following table. Occasionally, aggregates of such grading will be found that proportions

outside of the range in this table will be both desirable and economical.

Maximum Size of Coarse Aggregate, Inches	Ratio of Coarse to Fine on Basis of Dry Compact Volumes Minimum	Maximum
⅜	0.40	0.80
¾	0.60	1.50
1 and over	1.00	2.00

Production of Durable Concrete.

117. Q. *What are the principal requirements for durable concrete?*
A. The principal requirements for durable concrete are that the aggregates be durable and the concrete watertight.

118. Q. *How can durable aggregates be assured?*
A. By a knowledge of their past performance. Where only new and untried materials are available, an examination of the ledge or deposits may reveal the character of the material. In the absence of reliable indications as to the permanent character of a sample, it is best to consult a geologist before proceeding with its use in a structure intended for long service in severe exposure.

119. Q. *Why is watertight concrete necessary for durability?*
A. The greatest deterioration in exposed concrete comes from the penetration of moisture to the interior of the mass. Disintegration can be both chemical and physical; chemically, the action proceeds by the dissolving out of essential ingredients of the hardened cement paste; physically, by the action of frost on the entrained water or through the deposition near the surface of dissolved salts as the water is brought to the surface and evaporated.

120. Q. *What are the essential requirements for watertight concrete?*
A. The essential requirements for watertight concrete are durable aggregates completely incorporated in a cement paste that is itself impervious.

121. Q. *What are the requirements for an impervious cement paste?*
A. The requirements for an impervious cement paste are a low water-cement ratio and thorough curing.

122. Q. *How does the water-cement ratio affect the watertightness of the cement paste?*
A. As explained in the answer to Question 27, the amount of water used in mixing concrete is always far in excess of the amount that actually goes into combination with the cement, thus, the uncombined water occupies space in the mass which may later

form an opening for the passage of water. By keeping a low water-cement ratio the amount of this uncombined water is greatly reduced.

123. Q. *How does the curing of the concrete affect the watertightness of the cement paste?*

A. As explained in answers to Questions 5, 20, 25, and 26, continued curing builds up the internal structure of the cement paste so that it becomes more resistant to the penetration of moisture. Continued curing increases the amount of combined water and reduces the amount of uncombined water.

124. Q. *Is it possible to give values of the water-cement ratio for which watertight concrete can be assured?*

A. Not with the same degree of exactness that strengths can be related to water-cement ratio. The test data are very limited and the conditions of exposure are quite varied. It is possible, however, to give some limits which can be useful as a basis of design of concrete mixes for watertightness.

125. Q. *What water-cement ratios can be used with assurance that the concrete will be watertight?*

A. For concrete mixtures which will be placed in a plastic consistency and *thoroughly cured,* available tests and experience indicate that a water-cement ratio of about 6 gal. per sack should give a high degree of watertightness for ordinary structural requirements and severe exposure. For exposures to alkali waters and thin sections in sea water, about 5 to 5½ gal. of water per sack of cement would be a desirable maximum. For heavy walls, piers, dams, etc., water-cement ratios of 7 to 7½ gal. per sack have shown excellent results where the concrete has been properly placed and cured.

126. Q. *Why is the complete incorporation of the aggregates in the cement paste essential?*

A. If the paste is not sufficient in amount to actually float the aggregate particles, the mass will be harsh and granular, with unfilled spaces between the aggregate particles through which water may find its way.

127. Q. *How can this complete incorporation of the aggregates be assured?*

A. A homogeneous concrete mass with the aggregate particles thoroughly incorporated in the cement paste can be secured by using concrete of a plastic consistency carefully placed so as to completely fill the form without segregation or the accumulation of water at the surface.

128. Q. *What is the procedure for designing concrete mixes to meet a given condition of exposure?*

A. The procedure is exactly the same as in designing mixes to meet a strength requirement, except that the water-cement ratio is

selected to meet a given condition of exposure rather than a definite strength. The water-cement ratios given in the answer to Question 125 may be taken as a guide until more complete tables can be presented. For procedure, see answers to Questions 94 to 100 and succeeding questions.

129. Q. *What special precautions should be observed in the case of mixes designed for severe exposure?*

A. Particular care should be taken with the aggregate proportions to obtain a good plastic consistency. The utmost care should be taken in placing to obtain a homogeneous mass and in protecting to obtain thorough curing. It is also desirable to use somewhat longer mixing than for ordinary construction. (See answer to Question 52.)

PROBLEMS OF PLACING:

130. Q. *Why are dry mixes to be avoided?*

A. Dry mixes are difficult to place, making it hard to secure a structure that is free from voids or honeycomb.

131. Q. *Were not the structures generally successful, which were placed when dry mixes were the rule?*

A. Yes, but it was the practice to place the concrete in thin layers and thoroughly ram each layer until water appeared on the surface. By this method, it was possible to place concrete of a low water-cement ratio and still obtain a complete incorporation of all the aggregate particles in the cement paste. The flushing of water to the surface under the vigorous ramming was evidence that no unfilled voids remained. As pointed out in the answers to previous questions, to have the aggregate particles completely embedded in a cement paste of low water-cement ratio is the essence of durable concrete. It is because of the fulfillment of this requirement that so many of the early structures have been eminently successful. If the same careful ramming could be assured at the present time at a proper labor cost, the method would still be a desirable one in mass construction. The higher labor costs, combined with the use of reinforced concrete, have made the method practically obsolete.

132. Q. *Why are over-wet mixes to be avoided?*

A. Over-wet mixes segregate in handling and unless placed with extreme care, the thin watery mortar may escape leaving large stone pockets with no mortar filling. Such mixes, even when placed carefully, will settle in the form allowing the water to accumulate in the upper layers greatly reducing the strength and watertightness of these layers. The water-cement ratio-strength law does not hold for mixes of this type.

133. Q. *Are not over-wet mixes undesirable in reinforced concrete?*

A. The use of over-wet mixes in reinforced concrete work is particularly undesirable, as the settling of the heavier elements of the mix leaves water pockets on the underside of all rigidly fixed horizontal reinforcement bars. These water pockets, which greatly reduce the bond resistance, are greatest where the shear and bond stresses are most severe, near the points of maximum negative moment.

134. Q. *What is laitance?*

A. Laitance is an accumulation of the finer materials of the cement and aggregate which is brought to the top of a concrete mass by the use of excessively wet mixtures.

135. Q. *Is the laitance layer strong and resistant to weathering?*

A. No, the laitance layer possesses very little strength, and is rapidly disintegrated by the weather or the peneration of water.

136. Q. *Should the laitance layer be removed before placing concrete in a section above?*

A. Better still, it should never be allowed to form. Removing a layer of laitance only cures part of the evil. The very fact that laitance formed is evidence of grossly over-wet mixes, which means that the upper layers of concrete had a water content much higher than the average because of the accumulation near the top as mentioned in the answer to Question 132. Thus, removing only the laitance layer itself still leaves a mass of porous concrete immediately below, which will have low strength and may weather rapidly if ground or other water can find its way to it.

Not only should the laitance be removed, but several inches of the concrete below if a durable structure is desired.

137. Q. *How can laitance be avoided?*

A. By placing concrete of a plastic consistency that will not allow water to accumulate at the surface.

138. Q. *If there is a gradual accumulation of water at the surface as placing proceeds, what should be done?*

A. The batches should be gradually stiffened until at the top of the member or at the finish of the day's work, a proper plastic consistency has been restored.

139. Q. *How can the proportion of fine and coarse aggregate in the mix assist in proper placing?*

A. With a proper ratio of coarse to fine, a mix can be placed without difficulty that is somewhat stiffer than is possible where too high a proportion of coarse aggregate is used. This avoids both the necessity for wetter mixes and the danger of honeycombing due to harshness.

140. Q. With proper mix and proper consistency, is it still important to exercise care in the placing of concrete?

A. Yes, every detail of the placing should be controlled to insure the complete filling of the form and incorporation of the reinforcement with a mass of concrete that is free from voids or honeycomb and that is homogeneous from bottom to top. The practice of depositing the concrete all at one spot and allowing it to flow to the more distant points is particularly to be avoided as this is certain to result in some segregation no matter how carefully the mix and the consistency are controlled. The concrete should be deposited substantially in layers. In deep forms of large sections, the layers can be carried across the structure in such a way as to permit such water as comes to the surface to be drained to a point convenient for removal.

141. Q. What protection should be given concrete after it is placed?

A. Protection from drying out and from low temperatures during the early hardening period. The answers to Questions 19 to 34, also 121 to 123 should be reviewed. No detail of concrete construction offers such possibilities for increased strength and durability at so low a cost as are offered by the possibilities of better curing.

142. Q. Is good inspection an important feature of concrete construction?

A. Eternal vigilance is the price of success in concrete construction just as it has been the price of liberty in the history of mankind. Careful inspection should be enforced in all of the operations relating to:

The selection of the materials,
The design of the mixtures,
Mixing, transporting, and placing,
Protection and curing.

By careful and intelligent control at all of these stages, enduring structures can be achieved.

APPENDIX, CONCRETE PRIMER

DETERMINATION OF MOISTURE IN SAND.

* * * *

Of the different methods proposed for determining the moisture content of sand, the following two are probably the most simple and accurate:

(1) *Displacement Method with Cylindrical Container:*

Apparatus:

Balance sensitive to 1 gram.

Cylindrical container as shown in Fig. 6 with gage glass and scale calibrated to read to 5 cc.; 3 ft. of spring wire coiled at one end.

8-in. funnel with bottom diameter about 1½ or 2 in.

Method:

Fill cylindrical container with water up to zero mark on gage, insert wire into container allowing coiled end to rest on bottom. Pour a 2,000-gram sample of dry sand through funnel into container and gradually withdraw the wire, agitating the sand while so doing. Read the volume of water displaced. Repeat the operation using the same weight (2,000 grams) of the damp sand, whose moisture content is to be determined. The percentage of moisture may then be calculated from the formula:

$$p = 100 \frac{D - C}{W - D}$$

where p = percentage of moisture by weight of dry sample exclusive of absorbed moisture,

D = weight of water displaced by damp sand of weight W.

C = weight of water displaced by dry sample of weight W,

W = weight of sample (dry, surface dry, or damp) (2,000 grams).

It is necessary to use a dry sample to establish the constant C. If the specific gravity of the sand changes it will be necesary to make a new determination on a dry sample. The advantage of this method is that if the determination on the dry sample is made quickly, the result gives the free or surface moisture directly, thus requiring no correction for absorbed moisture.

(2) *Drying to Constant Weight with Denatured Alcohol:*

Apparatus:

Balance sensitive to 1 gram.
12 x 8 x 2-in. metal bread pan.
3/8-in. steel rod about 18 in. long.
1/2-pt. cup.
Denatured or wood alcohol.

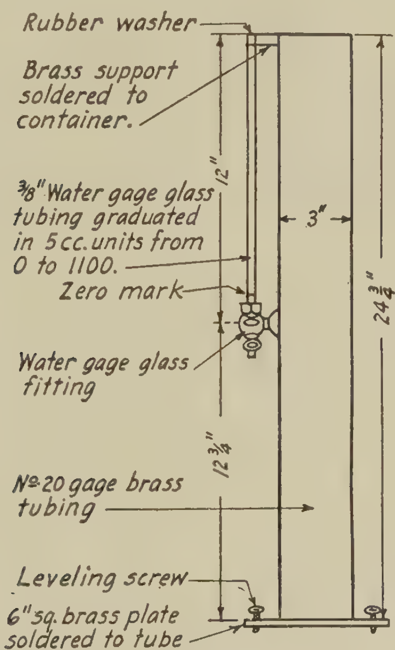


FIG. 6.—APPARATUS FOR DETERMINING MOISTURE IN AGGREGATE.

Method:

This method is the same as drying over a stove or open fire, but is much quicker and fully as accurate. Place 500 grams of damp sand in the bread pan. Pour 1/3 cupful of alcohol over the sand; stir the mixture with the rod and then spread in a thin layer over the bottom of the pan. Ignite the alcohol and allow it to burn until consumed, stirring the sand with the iron rod during burning. If the sand is excessively damp, it is advisable to repeat the burning process in order to insure complete drying of the sample. After burning, allow the sand to cool for 2 or 3 minutes

and then weigh. The total percentage, p, of moisture may then be calculated from the following formula:

$$p = 100 \frac{W - W'}{W'}$$

where p = percentage of moisture by weight of dry sample including absorbed moisture,
W = weight of damp sample,
W' = " " dry "

To determine the free moisture, it is necessary to know the percentage of moisture absorbed by the sand. This varies only slightly for different sands and may be assumed as 1 to 1½ per cent. A rough check on the absorbed water may be made by drying to constant weight a surface dry sample which had previously been immersed in water for a period of 24 hr. The surface moisture is then equal to the total moisture p minus the absorbed moisture.

INDEX

Discussion.—Concrete Primer.

Mr. Gilkey.

Herbert J. Gilkey.—The writer feels that Mr. McMillan is to be congratulated and commended upon this paper. He has recognized to an unusual and gratifying degree one of the greatest needs in the field of concrete. Much of our concrete literature has been directed toward no particular type of audience. Some of it has been frankly for specialists and is fully justified on that basis. Others of current offerings have been pseudo popu lar often admirable in spots but containing just enough of debated and debatable matter as to make them very confusing for those interested but not majoring in concrete.

The writer has felt that here has lain a great need, viz.: the setting forth in simple terms suitable for popular consumption those things about concrete regarding which there is general agreement and omitting entirely points that are still the subject of controversy and that need further investigation. The primer is quite conclusive evidence that Mr. McMillan has recognized the need and has therein made a long stride toward meeting it.

At the risk of wandering too far afield, the writer wishes to elaborate upon this point, for he feels that a better appreciation of it will greatly improve the status of concrete in more respects than one.

Concrete is an excellent illustration of a material that is of interest to several more or less distinct classes of people. For present purposes suppose we say three: (1) the investigator, (2) the user of concrete, (3) the bystander.

(1). *The investigator* is interested primarily in learning all that he can of the material. Because of the ease with which anyone can make and break a few specimens, this has been a numerous class and there have been all shades of investigative excellence. There are probably few attending this convention who do not at various times consider themselves to be bona fide concrete researchers. Some of us publish the results of apparent findings and among ourselves we freely accept, modify or reject one another's offerings. It is characteristic of the research frame of mind to be interested in truth for truth's sake. If two research men arrive at the same result by different routes, the fact that the ultimate conclusion is the same doesn't interest them. To be right for the wrong reason is to be still wrong and the seekers after knowledge will continue to fight it out on that line. And it is fitting and proper that they should. May the time never come when researchers are satisfied with unexplained results or with mutually incompatible explanations. For research will then be dead.

(2). *The user* is interested in results. He cares little or nothing about the underlying reasons; this quibble or that one. He does want approximate facts in the easiest shape for use.

(3). *The bystander* is the term used to designate all those not properly belonging to either of the above classes and yet who are likely to be wholly divorced from neither. The bystander is interested in concrete in a less detailed and critical manner than is the investigator, but his interest is not centered so exclusively upon the very practical aspects and immediate results as that of the user may be. This may include the average engineer or architect and certainly a great many other types of persons and professions. The bystander wishes to correctly know and understand concrete without becoming hopelessly enmeshed in the intricacies of divergent current theories upon the one hand, or yet be required to memorize a few rules of thumb upon the other.

In our zeal and eagerness to settle the fine points between theories and to spread varied gospels, we have rushed to the bystanders with our differences and, figuratively speaking, have made street brawls out of what should have been but family differences. Thus we who call ourselves investigators have monopolized the stage and the bystander has been sadly bewildered and amazed that the Almighty, with man's help, could have created a thing so complex as concrete. The user in turn has waited with impatience for verdicts from juries that never agreed, or better yet, has gone ahead and built an excellent structure while the wrangling continued.

By all means let the wrangling go on, for it is the essence of progress. But let us furnish a different sort of entertainment for those to whom the wrangling is but a source of bewilderment and confusion.

Of recent years, thanks to the splendid educational work of the Portland Cement Association and some others, occasional progress bulletins have been posted for the benefit of the user, and the results as reflected in the type of work now being turned out are testimonials to the avidity with which the users seize and apply any information handed to them in usable form.

Thus far the bystander has been most neglected. Each time he has been handed a morsel it was so flavored with one hobby or another that he dared not swallow it. One doctor told him it was the elixir of life and the next branded it as rank poison. As an engineering teacher, the writer has found no subject so difficult to teach as that of plain concrete. All existing literature of which he knows falls into one of two classes:

(a) It is thoroughly aligned to some one theory or set of theories and is thus either too narrow or too specialized.

(b) It attempts to be impartial and to give the essential points of all leading theories and proves to be such a confused medley as to leave the student in a most confused and hopeless state.

The present treatment is the most encouraging thing that the writer has seen along this line. Mr. McMillan doubtless has his own special ideas and theories. Who of us hasn't? These things are still being investigated.

In the meantime he has assembled a mass of matter regarding the most of which, at least, substantial agreement has been reached. Throughout the entire primer there are few statements to which any of us will take exception. Here is something that can be handed to the user and to the bystander and which will instruct without confusing, while we who wish to quibble may continue to do so back here in a star chamber. From out of this quibbling there will be evolved from time to time new matter for the primer.

Literature of this sort, free from fine points and technical differences should prove quieting to nerves overwrought through too much reading of rich or highly flavored concrete matter. The writer wonders if Mr. McMillan might not be able to evolve, from the present splendid beginning, a chapter upon plain concrete that could be substituted for this very unsatisfactory portion of most, if not all, of the present college text books on engineering materials.

Mr. McMillan. F. R. McMILLAN.—Perhaps a word or two in regard to the background of this Primer will be in order. The thought originated with the Board of Directors, and the title Primer and the presentation in catechism form was the suggestion of the Board. I was the goat to prepare the text.

In preparing the Primer I had to keep in mind some objective. I tried to set forth, in a limited series of questions and answers, some of the essential facts of concrete mixing and placing. There are many things we do not know about handling concrete. There are many things we do not know about the properties of cement or concrete, but there are certain things that have been fairly well established. In this little publication I have attempted to confine the text largely to those well-established principles. I have tried to arrange them in logical sequence, so that those who are unfamiliar with the vast amount of research and literature on concrete will form a fairly clear conception of what it is we are trying to do in concrete construction.

While it is called a Primer, perhaps it will not be for the primary class. At the beginning it seemed like that might be possible, but we had not gone far into the preparation of the text before it was realized that was not the field or exact purpose of the Primer. There are too many people in authority in concrete operation who are unable to keep themselves familiar with the vast literature on the subject and the meaning of it all. To the reader of that type, the question of concrete mixes has seemed a good deal of a mystery. It was my purpose to dispel a little of this mystery and to bring to the attention of the executive, the investor, the planning engineer, and the contractor, some of the things that are aimed at in these scientific studies.

In presenting this Primer, I do not claim to have covered the whole field; I do not claim to have treated completely the field that is covered. I have tried to condense as much as possible some of the essential things, and I believe it will be found helpful if it is studied. It is not a text to pick up and read a section here and there. It should be read through carefully by any one who wants to get the essence of what I am after.

C. M. CHAPMAN.—One of the Institute's functions is the dissemination of knowledge regarding concrete. New methods for the determination of some of the qualities of aggregates have been adopted by the American Society for Testing Materials and published in their proceedings for last year under Committee Reports and Tentative Standards. While these are available to you, it nevertheless might be well to give a little fuller description of these methods of tests and of the apparatus required. **Mr. Chapman.**

In Committee C-9 on Concrete Aggregates, of the American Society for Testing Materials, while discussing the water-cement ratio, the point was raised by some of the members that if you use the water-cement ratio you must allow for the surface moisture in the sand when measuring by damp volume or weight, or you must know the amount of water that fills the voids in your sand and so goes into the mixer with it if you measure by the inundation method. The statement was made that the water-cement ratio would not be generally adopted until we had a quick, easy method for determining these moisture conditions.

The Primer that has just been described contains in its appendix two very good methods for determining moisture but they do not apply to the determination of voids when the inundation method is used, nor do they apply to specific gravity. The American Society for Testing Materials has adopted as tentative standard, a flask of which I have here a sample, by means of which one may determine very quickly and easily the specific gravity, surface moisture and per cent of voids of fine aggregates. The flask consists essentially of two bulbs connected by a constricted neck and surmounted by a graduated tube. The volume of the lower bulb up to a mark on the constricted neck is exactly 200 cc., while the combined volume of the two bulbs up to the lowest graduation on the upper tube is 375 cc. The graduations on the upper tube are from 375 cc. to 450 cc.

To determine the specific gravity of fine aggregate fill the flask with water to the 200-cc. mark on the lower neck, then slowly pour in a 500-gr. sample of the aggregate to be tested and agitate by sudden rotation to free any entrained air bubbles. The sample must be free flowing but not thoroughly dry. Clean down any particles adhering to the tube of the flask, using the cleaner of which I have here a sample and which consists of a rod with a rubber disc on the end. Read the combined volume of the mixture on the graduated tube of the flask, and by means of a chart prepared for these determinations, find the specific gravity directly and without mathematical calculations.

The determination of surface moisture is made in a similar way, except that the sample shall truly represent the actual surface moisture content of the aggregate being used. Since the 500-gr. sample contains not only sand but also water, and that water being lighter than sand occupies a different volume per unit of weight, you will not get the same final reading on the flask as obtained in the specific gravity determination. Read the combined volume of the mixture as before, and from the chart and previously obtained specific gravity, read directly the per cent of surface moisture or gallons of water in 100 lb. of aggregate.

The voids determination is different. Fill the lower bulb of the flask about three-fourths full of water. Pour the material being tested into the flask until the level of both the water and the material reach the 400-cc. mark on the upper graduated tube of the flask. Add water during this operation as necessary to keep the level of the water just above the level of the material. When the level of the water and of the material are both at the 400-cc. mark, clean down the tube of the flask with the rubber disc cleaner. Weigh the flask and its contents to the nearest gram, deducting the weight of the empty flask, which is shown in grams on the upper bulb. From another chart read directly either the per cent of voids in the aggregate occupied by the water or the gallons of water in 1 cu. ft. of inundated aggregate.

Mr. Bergholm. A. O. BERGHOLM.—I am particularly interested in the determination of the moisture content of sand and I have run a few experiments. Say we get the surface moisture by the use of air drying, by heat drying or by the flask, I would like to know what the variations in the percentage of moisture would be by these three methods.

Mr. Chapman. C. M. CHAPMAN.—I am glad that Mr. Bergholm mentioned that point because it is one of the convenient features of the new flask. The amount of variation in the per cent of moisture when determined by each of the three methods mentioned will depend upon the character of the aggregate being tested. If you test a solid, dense, impervious silicious sand which has almost zero absorbtion, the results by all three methods will be quite similar. But if the aggregate is porous and has a high absorbtion factor, then the results will differ by the amount of the absorbed moisture. When using the water-cement ratio an allowance must be made for the moisture in the aggregate,—but only for the surface moisture—sometimes called free moisture. The moisture within the aggregate—the absorbed moisture—is not counted as a part of the water in figuring the water-cement ratio. The new flask determines only the surface moisture. If a sample is dried in an oven not only is the surface moisture removed but also all or part of the absorbed moisture, and the per cent of moisture determined by this method would be too high and a correction would have to be made before the result could be used in the water-cement ratio method. The amount of this correction would vary with different aggregates and would have to be determined for each aggregate. It would vary from zero to 3 or 4 per cent—in a few extreme cases even more. When the air-drying method is used, probably less of the absorbed moisture would be evaporated; the amount would vary with the surrounding air conditions and the time of drying. If the aggregate were air-dried only to a surface-dry or free-flowing condition, no correction would be necessary and the results would agree closely with the results obtained with the flask.

Mr. Stewart. G. M. STEWART.—May I describe a method which we have tried and which appears to be accurate? We determine the moisture by using the inundation process. First, we inundate a measure of dry sand to determine how much water it takes to perform the inundation. Then we inun-

date an equal amount of moist sand, and the difference is the moisture contained in the sand. The sand must be sun dried, not dried to drive off the absorption water. The advantage of the method is not so much in increased accuracy as in the fact that it can be done without any equipment other than a measuring can and a graduated flask for measuring the water.

We usually use a one-fifth cu. ft. measuring can. This is placed inside a tin pail, so that any water overflowing will be caught by the pail. After placing a known volume of water in the measuring pan, say 70 fluid oz., the dried sand is poured gently into the water until the measuring can is exactly full. A certain amount of dirty water overflows into the pail. The pan is then lifted out, the water poured into a flask and measured.

Suppose we get 15 oz. Since we started with 70 oz. the quantity required for inundation of the sand is 55 oz. We clean the apparatus and put 55 fluid oz. into the measuring can for the moist sand determination; perhaps we get 12 oz. of overflow. That was in the sand when we started; incidentally, we also determine the bulking of the sand in this way: Fill the measuring can with moist sand, dump it on to a piece of newspaper, fill the can a third full of water and pour the sand back again gently. Very little or no overflow will occur and the sand will settle down to a certain point in the can. I suggest this method tentatively because I do not know exactly how accurate it is, but it is a good deal better than guesswork.

W. A. SLATER.—The question has been asked about the agreement between the methods of determining moisture by drying and weighing and by an apparatus such as Mr. Chapman described. A few years ago I made some tests not with such an apparatus as that, but using the same principles of inundation. I do not recall exactly what the results were, but they were reported in the 1924 *Proceedings* of the American Concrete Institute in connection with a paper on the University of Illinois Stadium. **Mr. Slater.**

Design and Cost Data for the 1928 Joint Standard Building Code.

By Arthur R. Lord.*

SYNOPSIS.

The development of the technique of concrete proportioning within recent years, the constantly advancing knowledge of the mechanics of reinforced-concrete building design, the long years of study and research embodied in the 1924 report of the Joint Committee on Specifications for Concrete and Reinforced Concrete and the subsequent careful codification of that report by the Building Code Committee (E-1) of the American Concrete Institute has made available a workable and authoritative building code for all types of reinforced-concrete construction such as engineers in any city may adopt with confidence. One objection to such adoption lies in the loss of usefulness of most of the design tables and diagrams which have cost engineers a great deal in both time and money. To overcome this objection this paper includes a complete set of designers' tables and diagrams for use with the proposed 1928 Joint Standard Building Code. I believe that engineers will find this set of designers' aids as complete, as time-and-labor-saving and as accurate as any similar set they may be using under their local code. These tables and diagrams introduce important simplifications in the design of doubly-reinforced beams and in the spacing of stirrups. They cover a much wider range of concrete strengths than is covered by similar tables and diagrams previously published. Their use is illustrated and explained by numerous examples.

With these tables and diagrams as a foundation, a study has been made of the relative cost of the common types of structures using 2,000-lb. concrete as is now almost universal, except in columns, and using concrete of considerably greater ultimate strength. The advantage of higher strength concrete is indicated by many considerations. Our new control knowledge indicates that a fifty per cent increase in strength over the usual performance in concrete making in the past may readily be obtained with an increase in cost of about ten per cent. The use of higher strength concrete would also result in more workable concrete, less permeable and more highly resistant to the usual exposures to which outdoor concrete is subject. The attempt to utilize our newer knowledge of concrete propor-

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tioning to produce a better and a cheaper 2,000-lb. concrete is likely to result in harsh mixes and expensive placing. It seems to me better to use the same amount of cement, or somewhat more cement, with less water, and to employ in the design the higher strength so secured. The reduction in water will more than balance the addition of cement, insofar as shrinkage and crazing is concerned. This study indicates that a considerable direct saving in the final cost of an ordinary building is secured in addition to a higher quality of concrete and an easier field operation.

PART ONE—DESIGN DATA.

Introduction.—Almost from its founding, the American Concrete Institute has had a committee at work preparing and revising a building code which the Institute recommends as a standard, fairly representative of the best up-to-date knowledge and practice in reinforced-concrete design. This code is available to governmental agencies and has been adopted by a number of cities as well as by individual designers. The Institute has also been represented on the two National Joint Committees which have worked on this same problem on the *specification* side. I have been intimately connected with this work, as a member of the Institute's committee on building code for fourteen years, including five years as chairman, and as one of the Institute's representatives on the second Joint Committee. I have also participated professionally in the re-drafting of several city building codes and am now a member of the committee engaged in revising the Chicago code. The 1928 edition of the Institute's Standard Reinforced-Concrete Building Code is based on the 1924 Joint Committee report, in which the specification form has been changed to code form and in which scientifically accurate but practically cumbersome formulas and provisions have been simplified and made more workable as a basis of design and construction. In a few instances there are differences in the substance of the two reports but the greater part of the differences are entirely matters of form, adopted to make a *code* more acceptable to the engineers who must use and enforce it. In 1927 the Committee on Standard Practice of the Concrete Reinforcing Steel Institute which had previously adopted a code based on the 1924 Joint Committee report joined with Committee E-1 of the American Concrete Institute to formulate the 1928 Joint Code on which this paper is based. With respect to cost of buildings erected, this code is fairly representative of the more advanced general practice. Most city codes present certain sections which are far more liberal than the average national practice and in comparison with these sections the Joint Code would indicate added cost. On the other hand most city codes have other provisions which are unnecessarily burdensome and in these sections the Joint Code would show a saving. Comparisons with the Chicago code are included in Part II of the paper.

Some of the great differences in city codes have arisen from a feeling that the quality of the concrete in various localities must necessarily vary. We know today that a concrete of 2,000 or 3,000-lb. strength can be made

as readily in one city as in any other. Other differences in city codes have arisen from the political ascendancy of some business interest and represent merely inequitable variations in the factor of safety required in different members. The present situation in which 2,000-lb. concrete is limited to 650-lb. stress in one locality and forced to carry 1,200-lb. a few miles away, even though made from identical materials, is so absurd that the general adoption of a scientifically accurate and practically workable standard, such as the Joint Code, is a reasonable hope.

I have been using the substance of the 1928 Joint Code quite generally in my work since 1924, where city ordinances have not taken precedence, and as a result, prior to starting this paper, I had accumulated a considerable number of tables and diagrams to facilitate rapid and accurate design. I also recognized the need of still other designers' aids which time had not permitted me to develop. One great hardship involved in the adoption of the Joint Code by engineers generally would be the necessary scrapping of the old design tables and diagrams, on which much time and money had been spent, and the large new effort and expense which would be involved in creating in each office the necessary new diagrams and tables. This paper has been undertaken in the hope of supplying this need and of removing this obstacle. It represents also the conservation of a huge amount of duplicated effort which would otherwise be necessary. The preparation of the paper at this time has been made possible by aid from the Portland Cement Association, the Concrete Reinforcing Steel Institute and the Rail Steel Bar Association, making up the cost to me of producing the tables and diagrams not previously worked up and of making all of them conform strictly to the Joint Code as now presented.

Basis of the Tables and Diagrams.—In the preparation of these tables and diagrams, I have had in mind the importance of simplicity of presentation, in order to reduce both the time required for their application and the liability of error. Each diagram is based on a single steel stress and a single concrete stress, except that in some instances four parallel tables have been printed together to save space. In any practical design, only one set of stresses is involved and the work is facilitated and safeguarded if only this set is represented on the diagram. All stresses and provisions appearing in the paper correspond to my interpretation of the Joint Code. This code is unusually direct and understandable.

In the treatment of T-beams and of beams with compressive reinforcement I have adopted the device of reducing all designs to substantially the same process and method as is now universally used in the case of the rectangular beam. This method is based on the use of the full allowable stresses in both the steel and the concrete, in other words on the use of "balanced reinforcement." This has involved a large amount of computation to get accurate values of p and K for the great range of beam proportions used in design, but has greatly simplified and reduced the labor of design. By the tables presented here the usual involved diagrams are rendered unnecessary. A workable solution of the T-beam with compressive

reinforcement is presented for the first time and in the same simple manner. The unusual length of this paper has made it necessary to omit all formula derivation, but the formulas used are stated and figures are drawn in which the various dimensions and forces have been so represented as to aid anyone who wishes to check them. Every formula, table and diagram has been checked by some competent engineer other than myself and entirely outside of my own office.

In the design of certain types of members, such as footings or flat slabs, the proportions of the final structure may be varied through a considerable range and a large variety of equally correct designs may be secured, all complying with the Code. By a careful study of the average conditions encountered in practice it is possible—and customary in large offices—to set up standards of proportions to apply to all members of a given type. In such members all horizontal dimensions can be made to bear a constant ratio to the side of the floor panel or of the footing and all vertical dimensions a constant ratio to the depth of the slab or footing. In this way very simple diagrams can be prepared for "standardized" members and a vast amount of time and labor saved. This has been done in this paper and the "office standards" presented are fully described. In all such cases they are based on extended use in practice of substantially the same proportions.

The design of web reinforcement has been a thorn in the side of the concrete designer, with the result that wasteful guesswork has come to be all too common. The well-known shear diagram, which is easily sketched even for the most complicated cases as soon as the loads and reactions have been computed, has been used in this paper to compute directly and rapidly both the number of stirrups (vertical or inclined) and their accurate spacing. The diagrams perform for the designer the tedious and exacting work which has heretofore been required for accuracy and economy.

The work of the Division of Simplified Practice of the Department of Commerce in conjunction with the various trade organizations of the building industry has resulted in the elimination of much waste. In the field of reinforced concrete this work has resulted in the establishment of eleven standard bar sizes and of four standard sizes for spiral rods. Bar and spiral rod sizes which are no longer standard have been eliminated from my tables and diagrams. This will automatically eliminate the need for substitutions and back checking which would result from the accidental use of non-standard sizes, no longer commercially available. In the same way standard column capital sizes are used in one set of the flat slab diagrams.

Notation.—Standard notation as used in the Joint Code has been employed in all the text, tables, and diagrams of this paper. A few additional symbols have been used in some instances, and these are defined in the text.

General Tables.—Tables 1 and 2 are general tables, used in connection

with several types of members. They need very little explanation. Numerous other tables of this general group are available, but are not included when the results may be obtained by a simple slide rule operation and are usually so obtained by designers in preference to turning to the table.

Table 1 applies to the stem width of beams, T-beams, joists, etc., and its use will save computations involving both multiplication and addition.

Table 2 gives necessary information as to bars. Most designers know bar areas, but need to refer to such a table for values of $\sum o$ (= Summation of bar perimeters), in making bond computations. The value of $12a_s$ is useful in determining the bar spacing in one-way slab design.

Steps in Design for Flexure.—The design of all types of beams comprises seven steps, which are essentially the same for rectangular and T-beams with and without compressive reinforcement. These seven steps are stated completely under "Steps in Design of Rectangular Beams." In the other types of beams the slight differences occasioned by the introduction of special factors (t/d, d'/d, b' and p') are explained in full under each design step that is affected by them, while the steps that remain unchanged are not repeated on account of space limitations. The designer may not always be conscious of the individual steps but the process that he carries out is essentially that described below.

Steps in Design of Rectangular Beams.—(1) The size and weight of the member are assumed and the moments, shears and reactions are computed.

(2) The value of the effective depth, d, is assumed, the value of K for the concrete stress used in the design is taken from Table 3 or Table 4 and the value of b is computed by formula (101).

$$b = \frac{M}{Kd^2} \quad \text{.......} \quad (101)$$

(3) The value of v, the unit shearing stress, is computed by formula (102a).

$$v = \frac{8V}{7bd} \quad \text{.......} \quad (102a)$$

(4) The value of v must lie within the limits permitted by the code, which vary with the type of anchorage, and the weight of the beam including protective covering must agree with the weight assumed. If necessary, new assumptions must be made and the first four steps repeated.

(5) From the value of p for balanced reinforcement corresponding to the value of K used above, compute the area of tensile reinforcement by formula (103a).

$$A_s = pbd \quad \text{.......} \quad (103a)$$

(6) Select bars from Table 2 to make up the required area, checking the bond unit stresses by formula (17) of the code and the necessary stem width by Table 1.

(7) Complete by designing the web reinforcement.

Problem 1 shows the complete design of a rectangular beam by this common method while Fig. 3 gives the formulas relating to p and k, on which Tables 3 and 4 are based, and illustrates the stress relations.

Many cases will arise in design when it may be advantageous to maintain a uniform size of beam for several moment conditions, rather than to preserve "balanced reinforcement." In such instances the design will usually be made for the limiting beam from these tables and the concrete stresses in the other beams will be less than the full allowable value by the code. For these beams with reduced concrete stress, the steel area will be determined by formula (103b) in place of formula (103a) above.

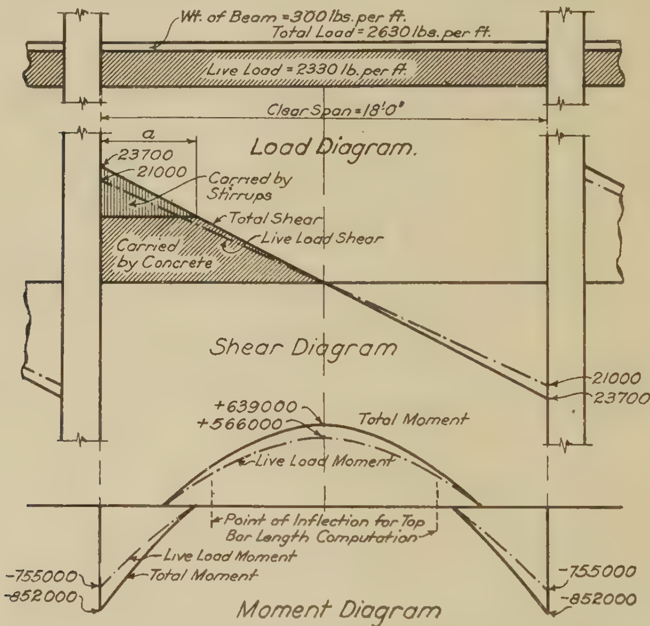


FIG. 1.—LOAD, SHEAR AND MOMENT DIAGRAMS FOR PROBLEM 1.

$$A_s = \frac{M}{17{,}500d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(103b)}$$

PROBLEM 1.

Having given the load, shear and moment curves for the superimposed uniformly distributed load as shown by the dash lines in Fig. 1 complete the design of the rectangular beam in accordance with the 1928 Joint Code, using 2,000-lb. concrete, fire resistive construction and deformed bars. The beam frames into reinforced-concrete columns at each end. The adjoining spans on either side are the same as this span.

Solution: Assume a beam 12 by 24 in. in section. The weight will be $(12)(24)(150)/144 = 300$ lb. per lin. ft. For this loading:

Dead-load moment at support $= wl^2/12 = \dfrac{(300)(18)(18)(12)^{(a)}}{12} =$ 97,000 in.-lb.

Total moment at support, $M_c = 97{,}000 + 755{,}000 = 852{,}000$ in.-lb.

Dead-load moment at center $= wl^2/16 = \dfrac{(300)(18)(18)(12)^{(a)}}{16} =$ 73,000 in.-lb.

Total moment at center, $M_c = 73{,}000 + 566{,}000 = 639{,}000$ in.-lb.

Dead-load reaction at each support $= (9)(300) = 2{,}700$ lb.

Total end shear $= 2{,}700 + 21{,}000 = 23{,}700$ lb.

The moment and shear curves, revised to include the dead-load, are shown in Fig. 1 by heavy full lines.

For fire resistive construction $d = 24 - (1\frac{1}{2} + \frac{1}{2}^{(b)} + \frac{5}{8}^{(c)}) = 21.4$ in. for assumed beam depth.

From the total end shear and formula (102a) $v = \dfrac{(8)(23{,}700)}{(7)(12)(21.4)} =$ 106 lb. per sq. in. ($= 0.053f'_c$). This is less than 120 lb. per sq. in. ($0.06f'_c$) and requires only ordinary anchorage.

At the support, in accordance with Table 4,

$$K = 157,\ p = 0.0091$$

By formula (101) $b = \dfrac{852{,}000}{(157)(21.4)^2} = 11.9$ in. (Against 12 in. assumed—O. K.)

By formula (103a)* $A_s = (0.0091)\ (11.9)\ (21.4) = 2.31$ sq. in.

At the center, in accordance with Table 3, $K = 131$, $p = 0.0075$

$$\text{By formula (101) } b = \frac{639{,}000}{(131)\ (21.4)^2} = 10.7 \text{ in.}$$

By formula (103a)* $A_s = (0.0075)\ (10.7)\ (21.4) = 1.71$ sq. in.

Several combinations of bars will satisfy this requirement. The arrangement that supplies the required steel areas both over the support and at the center with the least total cost of the steel in place should be used. In this determination the extras for bar sizes, the extras for bending or cutting small amounts of any one size and the extra labor cost of handling bars of many lengths varying by small amounts should all be considered.

Try 3-⅞ in. rd. at the center, bending up 1-⅞ in. rd. bar and lapping across the support. This will provide 2-⅞ in. rd. across the support ($= 1.20$ sq. in.) and 2-⅞ in. rd. straight in the top will make up the required 2.31 sq. in. From formula (17) of the code the bond stress on the lower steel at the point of inflection will be:

$$u = \frac{8\,V}{7\,\Sigma\,od} = \frac{(8)\,(14{,}200)^{(h)}}{(7)\,(5.5)^{(e)}\,(21.4)} = 138 \text{ lb. per sq. in. } (= 0.069f'_c)$$

*Formula (103b) may be used, as described on page 542.

This exceeds 100 lb. per sq. in. ($0.05f'_c$) and requires special anchorage for at least one-third of the center reinforcement. One ⅞ in. rd. must be carried into the support at each end a distance equal to $(16.7)^{(d)}$ (⅞) = 14.6 in. One ⅞ in. rd. bar will be carried to the center of the support. The bond stress on the upper steel at the edge of the support will be, by formula (17) of the code:

$$u = \frac{(8)(23{,}700)}{(7)(11.0)(21.4)} = 115 \text{ lb. per sq. in. } (= 0.058f'_c)$$

This exceeds 100 lb. per sq. in. ($0.05f'_c$) and requires special anchorage beyond the point of inflection (the fifth point of the clear span) of at least one-third of the steel. In this case the 2-⅞ in. rd. straight in the top must be carried $(16.7)_{(d)}$ (⅞) = 14.6 in. beyond the point of inflection or 4 ft. 10 in. beyond the face of the support.

In the shear diagram (Fig. 1) the vertically-hatched area representing the shear taken by the stirrups at either end is a *triangle,* which indicates that this is Case II (see p. 19) and the rapid solution for stirrup spacing given in Table 68 is available. The shear carried by the concrete by equation (114) is:

$$V_c = (60)^{(a)}(\tfrac{7}{8})(12)(21.4) = 13{,}500 \text{ lb.}$$

The distance, a, to the point where no web reinforcement is required, by formula (119) is:

$$a = \left(\frac{23{,}700 - 13{,}500}{23{,}700}\right)\left(\frac{(18)(12)}{2}\right) = 46.5 \text{ in.}$$

The area of the triangle under the shear curve is:

$$\Sigma V' = \left(\frac{23{,}700 - 13{,}500}{2}\right)(46.5) = 237{,}000 \text{ in. lb.}$$

The maximum permissible size of a vertical stirrup by Diagram 66 is ⅜ in. rd. for deformed stirrups. The total stirrup area at each end by formula (117) is:

$$NA_v = \frac{237{,}000}{(14{,}000)(21.4)} = 0.79 \text{ sq. in.}$$

From Table 67 we find that 4-⅜ in. rd. U-stirrups equal 0.88 sq. in.

From Table 68 for 4 stirrups and $a = 46.5$ in. we compute

Face of support to first stirrup = (.07) (46.5) = 3.25 in.	= 3¼ in.
Next two spaces = (.16) (46.5) = 7.44 in.	2 at 7½ in. = 15 in.
Last space = (.26) (46.5) = 12.1 in.	1 at 12 in. = 12 in.

By section 804, the maximum permissible stirrup spacing within the distance, a, is:

$$(0.75)(21.4) = 16.0 \text{ in.}$$

No extra stirrups are required by this limit. The spacing from the face of each support is:

3¼ in., 2 at 7½ in., 12 in.

The stirrup length is:

$$(12-3)+[(2)(21.4)]+[(2)(5)]=62 \text{ in.}=5 \text{ ft. } 2 \text{ in.}$$

A portion of the stirrups could be omitted in the zones where the bent-up bar reinforces the web. The omissions may be readily determined by drawing the stirrups and the bent bar to scale on the shear diagram as indicated in Fig. 12.

Notes for Problem 1:

[a] Multiplied by 12 to give moment in *inch*-pounds.
[b] Allowance made for ½ in. rd. stirrups—slightly excessive.
[c] Allowance made for 1¼ in. rd. bar—slightly excessive.
[d] By section 903 of the code:
Length of anchorage with deformed bars =

$$\frac{\left(\frac{20,000}{3}\right)\left(\frac{D^2}{4}\right)}{(0.05f'_c)(D)}=\frac{33,333D}{f'_c}=16.7D \text{ for 2,000-lb. concrete.}$$

[e] Σo for 2-⅞ in. rd. = 5.5 sq. in. from Table 2.
[f] Σo for 4-⅞ in. rd. = 11.0 sq. in. from Table 2.
[g] Since special anchorage has been provided to meet the bond requirements the value of v_c may be taken as $0.03f'_c$ or 60 lb. per sq. in. for 2,000-lb. concrete.
[h] Shear at point of inflection (fifth point of clear span) equals (0.6) (23,700) = 14,200.

Steps in Design of T-Beams.—T-beams, in which the slab acts as a compression flange for the beam stem, are the same as rectangular beams in which the values of p and K are reduced by the elimination of part of the section. In the analysis used in this paper, the compressive stresses in the stem between the neutral axis and the lower face of the flange are neglected, as is usual in design. In designing a T-beam the thickness, t, of the slab forming the flange has presumably been determined. The steps in the design are as follows:

(1) Same as rectangular beam except that only the weight of the *stem* of the beam need be assumed.

(2) The value of d is assumed, and from this the value of t/d is at once known. Enter Table 5 under the concrete stress used in the design and locate the value of K opposite the computed value of t/d. Record the value of p for use under step (5). From formula (101) determine the value of b.

(3) From formula (102a) rewritten with b', the stem width, in place of b, compute the value of v.

$$v=\frac{8V}{7b'd} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (102b)$$

(4) The value of v must be checked against the limits permitted by the code and the overhanging width of the slab used for the T-flange on either side of the stem must be checked also against the code. The weight of the stem including protective covering must be checked against the assumed weight.

(5), (6) and (7) are exactly the same as in the rectangular beam.

Problem 2 shows the complete design of a T-beam by this method while Fig. 5 gives the formula relating to p and K in such a beam, on which Table 5 is based, and illustrates the stress relations.

Steps in Design of Rectangular Beams with Compressive Reinforcement.—Compressive steel is introduced into a rectangular beam when the values of b and d are so limited by architectural considerations as to make the value of K from formula (104) greater than the value given in Table 3 (or Table 4) for the concrete stress permitted in the design.

$$K = \frac{M}{bd^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (104)$$

Since b and d are both known the design steps are modified as follows:

(1) The weight of the member is known and the moments, shears and reactions have been computed.

(2) Knowing d, the value of d'/d is closely established by the amount of covering required by the code. This should be taken as 0.02, 0.04, 0.06, etc., to 0.20 (whichever is the nearest to the value computed). Enter the appropriate table—8 to 15 inclusive—for the concrete stress used in the design and under the proper value of d'/d locate the value of K found by equation (104). Record the corresponding values of p and p' for use under step (5).

(3) and (4) are the same as in the rectangular beam.

(5) The area of the tensile reinforcement is found from formula (103a) and the area of the compressive reinforcement from formula (105).

$$A'_s = p'bd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (105)$$

(6) and (7) are the same as in the rectangular beam, with the added requirement of supplying the ties for the compressive steel.

In Tables 8 to 19 for special refinement of design the change in the values of p and K for small increments of compressive reinforcement are found by interpolation, permitting any degree of accuracy warranted by the balance of the design or the number of like beams involved. Problem 2 shows the complete design of a beam with compressive reinforcement by this simple method, while Fig. 5 gives the formulas relating to p, p' and K in such a beam, on which Tables 8 to 15 are based, and illustrates the stress relations.

PROBLEM 2.

The T-beams of a beam-and-girder floor are spaced six feet apart on centers and are supported on concrete girders spaced 24 ft. in the clear. The live-load and the weight of the 4-in. floor slab total 300 lb. per sq. ft. The adjoining spans on either side are the same as this span. Design the beam both at the center and at the support in accordance with the Joint Code, using 3,000-lb. concrete and fire resistive construction. The depth of the beam is limited by architectural considerations to a maximum of 16 inches.

Solution: Assume the beam stem as 12 in. wide. The weight of the stem, below the slab, will be (12)(12)(150)/144 = 150 lb. per lin. ft. The load from the slab will be (300)(6) = 1,800 lb. per lin. ft. and the total load 1,950 lb. per lin. ft.

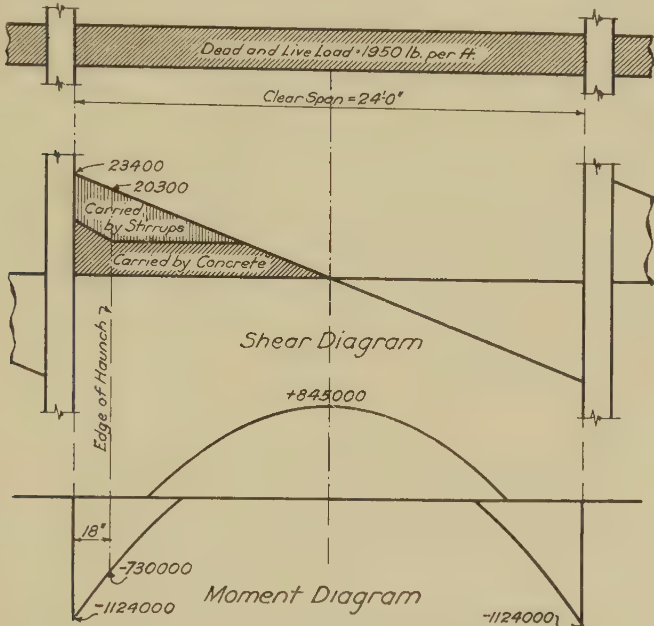


FIG. 2.—LOAD, SHEAR AND MOMENT DIAGRAMS FOR PROBLEM 2.

Design at center (T-beam).

Moment at center, $M_c = \dfrac{wl^2}{16} = \dfrac{(1{,}950)(24)^2(12)}{16} = 845{,}000$ in. lb.

$t = 4$ in. $\quad d = 16 - (1\frac{1}{2} + \frac{3}{8}{}^{(a)} + \frac{5}{8}{}^{(b)}) = 13.5$ in.

$t/d = 4/13.5 = 0.30$

From Table 5, for 3,000-lb. concrete, $p = 0.0108$ and $K = 192$

By Formula (101) $b = \dfrac{845{,}000}{(192)\ (13.5)^2} = 24.1$ in.

By Formula (103a)* $A_s = (0.0108)\ (24.1)\ (13.5) = 3.52$ sq. in. = 2-1⅛ in. sq. and 1-1 in. sq.

From Table 1, width of beam $= 9\frac{1}{2} + (\frac{3}{4}{}^{(a)} + 1^{(c)}) = 11\frac{1}{4}$ in. (12 in. assumed above—O. K.)

By formula (102b) $v = \dfrac{(8)(12)(1{,}950)}{(7)(12)(13.5)} = 165$ lb. per sq. in.

$(= 0.055 f'_c{}^{(e)})$

*Formula (103b) may be used, as described on page 542.

This is less than 180 lb. per sq. in. ($0.06f'_c$) and special anchorage is not required for diagonal tension. The overhanging flange width is only 6.1 in. on each side and is O. K. Assume that 1-1⅛ in. sq. bar will be bent up and the two remaining bars carried through in bottom.

From formula (17) of the code the bond stress at the point of inflection will be:

$$u = \frac{(8)\ (7.2)\ (1{,}950)}{(7)\ (8.5)^{(d)}(13.5)} = 140 \text{ lb. per sq. in. } (= 0.047f'_c)$$

This is less than 150 lb. per sq. in. ($0.05f'_c$) and only ordinary anchorage is required.

Design at Support (Rectangular Beam with Compressive Reinforcement).

Moment at support $M_s = \dfrac{wl^2}{12} = \dfrac{(1{,}950)\ (24)^2(12)}{12} = 1{,}124{,}000$ in. lb.

$b = 12$ in. and $d = 13.5$ in. as determined above, $d' = 2.7$ in.

$$d'/d = \frac{2.7}{13.5} = 0.2$$

By formula (104) $K = \dfrac{1{,}124{,}000}{(12)\ (13.5)^2} = 513$

An examination of Table 13 shows that K is too great and that the beam must be widened at the support. Try $b = 20$ in.

By formula (104) $K = \dfrac{1{,}124{,}000}{(20)\ (13.5)^2} = 308$

From Table 13 using $d'/d = 0.2$; for $K = 313$; $p' = 0.016$ and $p = 0.0185$.

By formula (105) $A' = (0.016)\ (20)\ (13.5) = 4.32$ sq. in.

If the 1-1⅛ in. sq. and 1-1 in. sq. in the bottom are lapped, an area of 4.53 sq. in. will be provided.

By formula (103a)* $A_s = (0.0185)\ (20)\ (13.5) = 5.00$ sq. in.

The 1⅛ in. sq. bent up from the center and lapped across the support provides 2.53 sq. in. Two 1⅛ in. sq. straight in the top will provide 2.53 sq. in. or a total of 5.06 sq. in.

The critical bond stress at the face of the support will be:

$$u = \frac{(8)\ (12)\ (1950)}{(7)\ (18.0)^{(f)}\ (13.5)} = 110 \text{ lb. per sq. in. } (= 0.037f'_c) \text{ requiring}$$

ordinary anchorage only.

The increase of b to 20 in. at the support must be tapered down to the center width of 12 in. not nearer the support than that point where the moment is reduced to the value of the resisting moment of the doubly

*Formula (103b) may be used, as described on page 542.

reinforced 12 by 16-in. section. This may generally be assumed from an inspection of the moment curve but may be computed as follows:

For 2—1⅛ in sq. and 2—1 in. sq. in bottom, $b = 12$ in. and $d = 13.5$ in.; $p' = 0.028$

For 4—1⅛ in. sq. in top, $b = 12$ in. and $d = 13.5$ in.; $p = 0.032$

The code permits only 2 per cent of compressive reinforcement to be considered effective and with $d'/d = 0.2$ the value of K from Table 13 is limited to 333. The maximum resisting moment of the 12 by 16 in. section is therefore:

$$M = (333)(12)(13.5)^2 = 730{,}000 \text{ in. lb.}$$

The haunch may be terminated at a point 18 in.[g] from the face of the support where this negative moment is shown by Fig. 2.

The unit shearing stress has been decreased by the increase in b. At the support by formula (102a):

$$v = \frac{(8)(12)(1950)}{(7)(20)(13.5)} = 99 \text{ lb. per sq. in.}$$

At the end of the haunch, by formula (102b).

$$v = \frac{(8)(10.5)(1950)}{(7)(12)(13.5)} = 145 \text{ lb. per sq. in. } (= 0.048 f'_c)$$

Only ordinary anchorage is required for either bond or diagonal tension and the value of v_c is $0.02 f'_c = 60$ lb. per sq. in.

By formula (114) $V_c = (60)(⅞)(12)(13.5) = 8{,}500$ lb.

At face of support $V_c = (60)(⅞)(20)(13.5) = 14{,}200$ lb.

Compute the stirrups from the narrow end of the haunch toward the center of the beam as in Problem 1 and continue the next-to-end spacing for stirrups within the haunch. Formula (119) determines the value of a and is not affected by the haunch. In this case the stirrups are more than adequate to meet the code requirements for ties for the compressive reinforcement.

Notes for Problem 2.

(a) Allowance made for ⅜ in. stirrups.
(b) Allowance made for 1¼ in. bar.
(c) Allowance made for fire resistive construction.
(d) Σ o for 1—1⅛ in. sq. and 1—1 in. sq. bars = 8.5 sq. in. from Table 2.
(e) Note change in beam width and recalculation of v later in problem.
(f) Σ o for 4—1⅛ in. sq. bars = 18.0 sq. in. from Table 2.
(g) May be computed as follows:

$$M_x = 1{,}124{,}000 - 730{,}000 = 394{,}000 \text{ in. lb.} = 32{,}800 \text{ ft. lb.}$$

$$= \frac{wx}{2}(l - x) = 975x(24 - x)$$

Solving: $x = 1.50$ ft. = 18 in.

Steps in Design of T-Beams With Compressive Reinforcement.— T-beams may occasionally require compressive reinforcement in order to comply with the code limitations of b and still keep within an architec-

tural limitation on the beam depth or flange width. This case is slightly more complex but still simple under this method of solution. The limiting values of b, d and t/d will be known. The steps are as follows:

(1) Same as for ordinary T-beam, except that weight of beam stem is definitely determined.

(2) From the values of b and d compute the required value of K by formula (104). Enter Table 16 (or Table 17, 18 or 19 according to the concrete stress used in the design) and record the value of p and K for the known value of t/d, using the three left-hand columns of the table. Subtract this K, which is the value for a T-beam without compressive reinforcement, from the total K found from formula (104). The remainder is the value of K which must be added by the compressive reinforcement and its balancing additional tensile reinforcement. Under the proper value of d'/d locate the value of K equal to this remainder and record the corresponding values of p and p'. Add the two values of p together to get the total percentage of tensile reinforcement for the beam.

(3) Same as ordinary T-beam.

(4) Same as ordinary T-beam except that the values of b and of the weight need not be checked.

(5) Compute the area of tensile reinforcement from formula (103a) and of the compressive reinforcement from formula (105).

(6) and (7) are the same as in the rectangular beam. In Tables 16 to 19 the change in the values of p and K for small increment of compressive reinforcement are given at the bottom, permitting very accurate interpolation where conditions warrant.

Problem 3 shows the complete design of a T-beam with compressive reinforcement by this simple method. Fig. 6 gives the formulas relating to p, p' and K, on which Tables 16 to 19 are based and illustrates the stress relations.

PROBLEM 3.

A floor has standard pans 8 in. deep by 20 in. wide and 2-in. concrete top slab. The clear span between joist supports is 20 feet.

Live Load $= (2.08)(50)$	$= 104$
Top Slab $= (2)(25)$	$= 50$
Joist Stem $= \left(\frac{5+7}{2}\right)(8)$	$= 48$
Susp. Ceiling $= (2.08)(10)$	$= 21$
Wood Floor on Fill $= (2.08)(20)$	$= 42$
Partitions $= (2.08)(35)$	$= 73$
Total Load per lin. ft.	$= 338$

The loads on the typical joist are as given to the left, and the design provides joists 5 in. wide and 25 in. on centers. A storage room requires a number of joists to be designed for a live load of 250 lb. per sq. ft., the other loads remaining as before except that the partition load may be considered as included in the heavier live load. The depth of the construction must remain 10 in. as before for architectural effect on the ceiling of the room below. Design these special joists, using compressive reinforcement if necessary, with 2,000-lb. concrete.

Solution: For the typical joists the design at the center is summarized as follows:

$$d = 10 - (1^{(a)} + \tfrac{1}{4}^{(b)} + \tfrac{1}{2}^{(c)}) = 8.25 \text{ in. } t/d = \frac{2}{8.25} = 0.24$$

From Table 5, for 2,000-lb. concrete, $p = 0.0065$ and $K = 117$

$b = \dfrac{(338)(20)^2(12)}{(16)(117)(8.25)^2} = 12.7$ in. $A_s = (0.0065)(12.7)(8.25) = 0.68$ sq. in. $= 1\text{-}\tfrac{3}{4}$ in. rd. and $1\text{-}\tfrac{1}{2}$ in. sq.

Width of joist from Table 1 $= 4\tfrac{7}{16} + \tfrac{1}{2}^{(b)} = 4\tfrac{15}{16}$ in. (5 in. used).

The loading on the store room joists will be increased by (250) (2.08) $= 520$ lb. per lin. ft. and decreased by (2.08) (85) $= 177$ lb. per lin. ft., the net increase being 343 lb. per lin. ft. and the final load on these joists 681 lb. per lin. ft. For this load:

$b = \dfrac{(681)(20)^2(12)}{(12)^{(d)}(117)(8.25)^2} = 34.3$ in. This is greater than the available width of 25 in. and compressive reinforcement is required.

Design of T-Joist with Compressive Reinforcement

$$\text{Moment at Center} = \frac{(681)(20)^2(12)}{12^{(d)}} = 272{,}400 \text{ in. lb.}$$

$b = 25$ in. $d = 8.25$ in. Therefore $K = \dfrac{272{,}400}{(25)(8.25)^2} = 160$

From Table 16, left portion, with $t/d = \dfrac{2}{8.25} = 0.24$, $K = 117$ and $p = 0.0065$.

The deficiency in K is therefore $160 - 117 = 43$

$$d' = 1^{(a)} + \tfrac{1}{4}^{(b)} + \tfrac{3}{8}^{(c)} = 1\tfrac{5}{8} \text{ in.}; \quad d'/d = \frac{1.63}{8.25} = 0.2$$

From Table 16, right portion, using $d'/d = 0.2$ we find that $K = 42$ when $p' = 0.010$ and $p = 0.0026$.

The total percentage of tensile steel is $0.0065 + 0.0026 = 0.0091$.

By formula (103a)* $A_s = (0.0091)\ (25)\ (8.25) = 1.88$ sq. in.

This requires a joist about 8 in. wide and the design must be revised for the extra dead- and live-load, as follows:

Live load $= (2.33)\ (250) = 583$. Mom. at center $= \dfrac{(781)(20)^2(12)}{12}$

Top Slab $= (28)\ (2) = 56$. $= 312{,}400$ in. lb.

Joist stem $= \left(\dfrac{8+10}{2}\right)(8) = 72$. $b = 28$ in. $d = 8.25$ in. $K = \dfrac{312{,}400}{(28)(8.25)^2}$ $= 164$.

*Formula (103b) may be used, as described on page 542.

Susp. Ceiling = (2.33) (10) = 23.
Wood Floor on fill = (2.33) (20) = 47.
Total load per lin. ft. = 781.

Deficiency in $K = 164 - 117 = 47$.
From Table 16, $p' = 0.012$ and $p = 0.0031$.

The total percentage of tensile reinforcement is:

$$p = 0.0065 + 0.0031 = 0.0096.$$

By formula (103a)* $A_s = (0.0096)(28)(8.25) = 2.22$ sq. in. = 2 — 1 in. rd. and 1-⅞ in. rd. in bottom.

From Table 1, required width of joist $= 5\frac{1}{2} + 2^3/_{16} + \frac{1}{2} = 8^3/_{16}$ in. [(e)]

By formula (105) $A'_s = (0.0031)(28)(8.25) = 0.72$ sq. in. = 1-¾ in. rd. and 1-⅝ in. rd. in top.

By formula (102a) $$v = \frac{(8)(10)(781)}{(7)(8.25)^{(e)}(8.25)} = 131 = 0.066f'_c.$$

This requires special anchorage of reinforcement. The design of the joist at the support and of the web reinforcement, follows the same procedure as shown in Problem 2, and will not be repeated here. The flared joist furnished by tapered end-pans will provide the necessary joist width at the support with compressive reinforcement and will reduce the shearing unit stress.

The deflection should be investigated and a camber placed in the forms to equalize the final deflection of the typical and special joists.

For the typical joist, $$D = (0.0625)\left(\frac{(240)^2}{8.25}\right)(0.00092) = 0.40 \text{ in.}$$

For the special joist, $$D = (0.0833)\left(\frac{(240)^2}{8.25}\right)(0.00107) = 0.62 \text{ in.}$$

The formwork for the special joists should be cambered 0.22 in. more than that for the typical joists in order to keep a level ceiling.

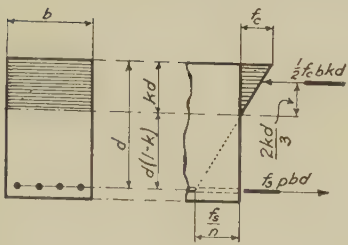
Notes for Problem 3.

(a) Allowance made for ordinary construction.
(b) Allowance made for ¼ in. rd. stirrups.
(c) Allowance made for one-half of ¾ in. bar.
(d) Moment at center increased to $\frac{wl^2}{12}$ on account of special loading.
(e) Width of joist at level of bottom reinforcement = 8¼ in.

Combined Bending and Direct Compression.—I have developed no new methods for the design of members subject to bending and direct compression. In general, it is not possible to maintain "balanced reinforcement" in such designs and each diagram must cover a considerable range of stresses both in steel and in the concrete. Diagrams of this same nature. that have been published heretofore, have generally covered too narrow a

*Formula (103b) may be used, as described on page 542.

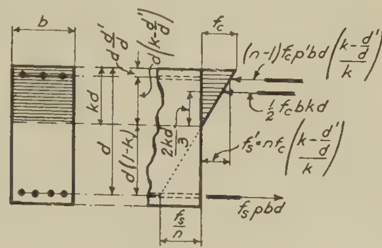
FORMULAS FOR DESIGN VALUES OF p AND K.



$$p = \frac{f_c}{f_s} \cdot \frac{k}{2}$$

$$K = f_c \left(\frac{k}{2} - \frac{k^2}{6}\right)$$

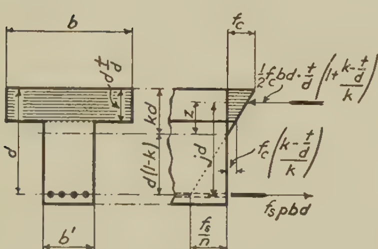
FIG. 3.—RECTANGULAR BEAMS.



$$p = \frac{f_c}{f_s}\left[\frac{k}{2} + \frac{n-1}{k}\left(k - \frac{d'}{d}\right)p'\right]$$

$$K = f_c\left\{\frac{k}{2} - \frac{k^2}{6} + \frac{n-1}{k}p'\left[(1-k)\left(k-\frac{d'}{d}\right) + \left(k - \frac{d'}{d}\right)^2\right]\right\}$$

FIG. 5.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

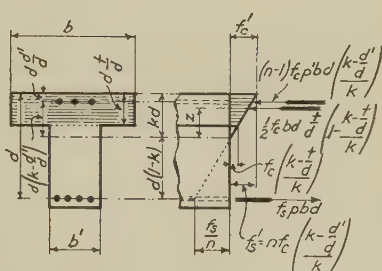


$$z = d\left[k - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

$$p = \frac{f_c}{f_s} \cdot \frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)$$

$$K = f_c \cdot \frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)\left[1 - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

FIG. 4.—T-BEAMS.



$$z = d\left[k - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]$$

$$p = \frac{f_c}{f_s}\left[\frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right) + \left(\frac{n-1}{k}\right)\left(k - \frac{d'}{d}\right)p'\right]$$

$$K = f_c\left\{\frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)\left[1 - \frac{t}{d}\left(\frac{3k - 2\frac{t}{d}}{6k - 3\frac{t}{d}}\right)\right]\right.$$

$$\left. + \left(\frac{n-1}{k}\right)p'\left[(1-k)\left(k - \frac{d'}{d}\right) + \left(k - \frac{d'}{d}\right)^2\right]\right\}$$

FIG. 6.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT.

range to be fully adequate for design. Diagrams 20 to 65 cover both circular and rectangular sections for Cases I and II to the full limits of steel ratio permitted by the code, for 2,000, 2,500, 3,000 and 3,750-lb. concrete (and for 5,000-lb. concrete in circular sections) and for ratios of d'/t up to 0.2. All these diagrams have been newly computed for this paper. So far as I know, this is the first time that diagrams for 3,000, 3,750 and 5,000-lb. concrete have been prepared. The essential formulas are given below and Problems 4, 5 and 6 illustrate their application. For the derivation of the formulas and a complete descriptive text the treatment of this subject in "Structural Members and Connections," pages 526 to 567, is recommended as probably the most complete of the available discussions.

Special Notations for Bending and Direct Compression.—New symbols, as used in the formulas, diagrams and problems, have the following significance:

e = distance from point of application of N to gravity axis of section.

N = component, normal to section, of all forces acting on it.

L, Q, Z, R_1 = expressions introduced to reduce labor of computations.

p_0 = ratio of total area of symmetrically placed reinforcement to gross area of rectangular sections or to core area of circular columns.

r = distance of c. g. of steel area near compressive face of rectangular section to gravity axis.

r = radius of circular core.

t = total depth of rectangular section.

Design Formulas for Bending and Direct Compression.—For either round or rectangular sections the condition in which the entire cross section is in compression is designated as Case I. The condition in which part of the section is in tension is designated as Case II. Only symmetrical arrangement of reinforcement is covered by the design diagrams and the steel must be equally divided between the two faces in the case of rectangular sections. The usual design formulas follow:

Round Sections—Case I.

$$\text{Maximum stress in concrete, } f_c = \frac{NQ}{\pi r^2} \quad \text{.........(106)}$$

The value of Q is taken from Diagrams 20, 21, 22, 23 or 24.

*Round Sections—Case II.**

$$R_1 = \frac{Ne}{\pi r^3} \quad \text{.....................................(107)}$$

$$\text{Stress in concrete, } f_c = \frac{R_1}{\frac{R_1}{f_c}} \quad \text{..................(108)}$$

* Formulas for circular sections are based on a solution by Mr. C. S. Whitney, Consulting Engineer, Milwaukee, Wisconsin.

Tensile stress on steel, $f_s = R_1\left(\frac{f_s}{R_1}\right)$ .... (109)

Values of $\frac{R_1}{f_c}$ and of $\frac{f_s}{R_1}$ are taken from Diagrams 25, 26, 27, 28 or 29. Problem 4 illustrates the application of these formulas and diagrams to design.

For rectangular sections it must first be determined whether the solution falls under Case I or Case II. Case I formulas and diagrams must be used when the existing value of e/t is less than the value of e/t computed from the formula (110).

$$\frac{e}{t} = \frac{1 + 12np_o\left(\frac{r}{t}\right)^2}{6(1 + np_o)} \qquad \text{(110)}$$

Case II formulas and diagrams must be used when it is greater.

Rectangular Sections—Case I.

Maximum stress in concrete, $f_c = \frac{NZ}{bt}$ .... (111)

The value of Z is taken from Diagrams 30 to 45 for various values of d'/t from 0.05 to 0.2. Problem 5 illustrates the application of this formula and these diagrams to design.

Rectangular Sections—Case II.

Stress in concrete, $f_c = \frac{Ne}{bt^2L}$ .... (112)

Tensile stress in steel, $f_s = nf_c\left(\frac{d}{kt} - 1\right)$ ...... (113)

The value of L is taken from Diagrams 50, 55, 60 or 65 after the value of k has first been determined from Diagrams 46 to 49, 51 to 54, 56 to 59 or 61 to 64. Problem 6 illustrates the application of these formulas and diagrams to design.

The formulas for Q, Z and L may be found in the text referred to above as well as a full statement of the steps to be taken in design and formulas for the general case of non-symmetrical reinforcement for which complete design diagrams are not available.

PROBLEM 4.

A spiral column carrying 400,000 lb. of direct load including its own weight, is also subject to 4,800,000 in. lb. bending moment. Design the column in accordance with the 1928 Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: Assume a 36-in. diameter column with a 32-in. round core and $p = 0.04$.

$$e = \frac{4{,}800{,}000}{400{,}000} = 12 \text{ in.} \qquad \frac{r}{e} = \frac{16}{12} = 1.33$$

This is obviously Case II, since e is so large.

From Diagram 27, upper portion: $R_1/f_c = 0.296$.

From Diagram 27, lower portion: $f_s/R_1 = 18.0$

By formula (107) $R_1 = \dfrac{M}{\pi r^3} = \dfrac{4{,}800{,}000}{(3.14)(16)^3} = 373$

By formula (108) $f_c = \dfrac{373}{0.296} = 1{,}260$ lb. per sq. in.

(1,350 lb. per sq. in. allowed adjacent to support)

By formula (109) $f_s = (18.0)(373) = 6{,}720$ lb. per sq. in.

The core area by Table 105 is 804.2 sq. in.

The steel area will be:

$A_s = (0.4)(804.2) = 32.2$ sq. in. $= 21\text{-}1\frac{1}{4}$ in. sq. bars spaced uniformly inside the spiral.

The spiral will be determined by the design of this same column for the *direct load* only by the method shown in Problem 9.

PROBLEM 5.

A rectangular tied column, whose lesser dimension is limited to 20 in. carries a direct load of 800,000 lb. including its own weight and is also subject to a bending moment of 2,400,000 in. lb. acting in a plane perpendicular to the longer dimension of the column. Design the column in accordance with the 1928 Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: From the limit given in the problem, $t = 20$ in. Assume $p = 0.04$ and $d' = 0.15t$, making $r = 0.35t$.

$$e = \frac{2{,}400{,}000}{800{,}000} = 3 \text{ in.} \qquad \frac{e}{t} = \frac{3}{20} = 0.15$$

By formula (110) $\dfrac{e}{t} = \dfrac{1 + (120)(.04)(.35)^2}{6 + (60)(.04)} = .189$

This computed value of $\dfrac{e}{t}$ is greater than the existing value by the conditions of the problem and the diagram for Case I (Compression over entire section) apply.

From Diagram 40, $Z = 1.28$.

By formula (111) $b = \dfrac{(800{,}000)(1.28)}{(1{,}350)(20)} = 37.9$ in.

The concrete stress is 1,350 lb. per sq. in. as used in solving formula (111) above. The steel stress will be low since no tension occurs.

The column will be 38 in. wide by 20 in. deep.

The steel area will be:

$A_s = (0.04\ (37.9)\ (24) = 30.4$ sq. in. $= 20\text{-}1\frac{1}{4}$ in. sq. bars placed symmetrically with ten $1\frac{1}{4}$ in. sq. bars at each face.

The value of d' will be $2 + \frac{5}{8} = 2.62$ in. and $d' = 0.131t$ which checks the value assumed.

The ties will be $\frac{1}{4}$-in. rd. at 12 in. o.c. and provided with one leg through the center of the column in each direction in addition to the outer tie enclosing all the bars.

PROBLEM 6.

A square tied column carries a direct load of 200,000 lb. including its own weight and is also subject to a bending moment of 4,000,000 in.-lb. Design the column in accordance with the Joint Code with 3,000-lb. concrete, and determine the principal stresses.

Solution: Assume a column 26 in. by 26 in. and assume $p = .03$. By this assumption, $t = 26$ in. and $d' = 2.6$ in. $= 0.1t$ allowing 2-in. fireproofing.

$$e = \frac{4{,}000{,}000}{200{,}000} = 20 \text{ in.} \qquad \frac{e}{t} = \frac{20}{26} = 0.77$$

By formula (110) $$\frac{e}{t} = \frac{1 + (120)\,(.03)\,(.40)^2}{6 + (60)\,(.03)} = 0.20$$

This is Case II, since 0.20 is less than 0.77

From Diagram 57, with $\frac{e}{t} = 0.77$ and $p = .03$; $k = 0.49$

From Diagram 60, Part I, with $p = .03$ and $k = 0.49$; $L = 0.18$

By formula (112) $$f_c = \frac{4{,}000{,}000}{(26)\,(26)^2(0.18)} = 1{,}263 \text{ lb. per sq. in.}$$

A slightly smaller value of p may be determined by trial to give a higher concrete stress if it is desired to use the full 1,350 lb. per sq. in. allowed at section adjacent to the support.

The steel area will be:

$A_s = (.03)\,(26)^2 = 20.3$ sq. in. $= 13\text{-}1\frac{1}{4}$ in. sq. bars

Since the concrete stress is low we may use $12\text{-}1\frac{1}{4}$ in. sq. bars and secure a symmetrical arrangement of the bars.

By formula (113) the tensile stress in the steel will be:

$$f_s = (10)\,(1{,}263)\left[\frac{23.4}{(0.49)\,(26)} - 1\right] = 10{,}550 \text{ lb. per sq. in.}$$

Web Reinforcement—General.—The last step in the calculations for each of the four types of beams calls for the design of the web reinforcement. In this paper all formulas for the design of web reinforcement are

BASIS OF FORMULAS FOR WEB REINFORCEMENT.

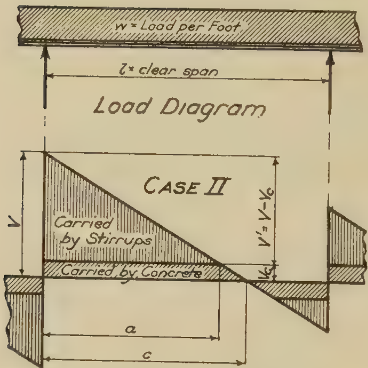


FIG. 7.—SHEAR DIAGRAM FOR UNIFORM LOAD ONLY.

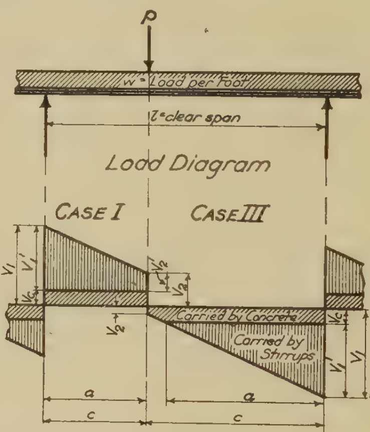


FIG. 9.—SHEAR DIAGRAMS WITH CONCENTRATED AND UNIFORM LOADS.

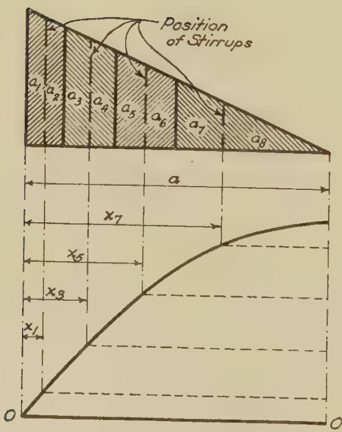


FIG. 8.—STIRRUP SPACING FOR CASES II AND III.

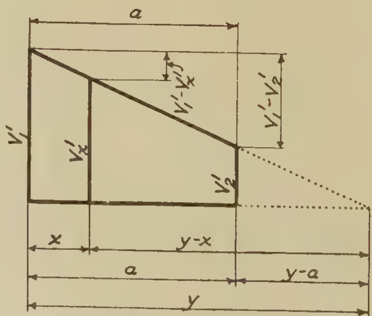


FIG. 10.—STIRRUP SPACING FOR CASE I (ENLARGED FROM FIG. 9).

based on the vertical shears in the usual manner. The web reinforcement computed from them will be adequate in amount and properly arranged to resist the diagonal tensile stresses which occur in the web. In Fig. 7 and Fig. 9 are shown the three shapes of areas that may occur in the shear diagram between the axis and the shear curve. Each area represents the total shear to be resisted by the web concrete and reinforcement. In Fig. 7, with uniform load only on the beam, these areas are shown to be triangles. This is designated as Case II and is a special form of the general trapezoidal case. In Fig. 9, with both uniform and concentrated loads on the span, these areas are trapezoids. At the left, the minimum shear at the low end of the trapezoid is greater than V_c, the safe resistance of unreinforced web concrete. This is designated as Case I. On the right the minimum shear at the low end is less than V_c. This is designated as Case III. Fig. 8 illustrates the law governing stirrup spacing. Fig. 10 is introduced to aid anyone who wishes to check the formulas in the following treatment but is of no interest otherwise.

In design by the Joint Code, the area under the shear curve is considered as made up of two parts. The first part is the resistance of the concrete itself at the unit shearing stress permitted by Section 306 of the Code. This shear resistance at any section is determined by formula (114).

$$V_c = \frac{7}{8} v_c \, bd \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (114)$$

In Figs. 7 and 9, this value of the shear on the concrete, V'_c is laid off and a line drawn parallel to the axis, defining the area (shown by diagonal cross-hatching) which represents the portion of the total area under the shear curve which may be considered as carried by the concrete without the aid of web reinforcement. This leaves the second part of the area under the shear curve (shown in Figs. 7 and 9 by vertical cross-hatching) as the measure of the shear resistance which must be provided by the web reinforcement.

For vertical stirrups, the total resistance, $\Sigma V'$, with N stirrups, is found by formula (115).

$$\Sigma V' = \frac{7}{8} N A_v f_v d \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (115)$$

For inclined stirrups, the total resistance, $\Sigma V'$, is found by formula (116).

$$\Sigma V' = \frac{7}{8} N A_v f_v d \text{ cosec } \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (116)$$

The value of $\Sigma V'$ is the vertically-hatched area under the shear diagram and should be computed taking the shear in pounds and the distance along the axis in inches. From this, the number and size of stirrups may be computed by formulas (115) and (116). Using the stirrup steel stress permitted by the code (16,000 lb. per sq. in.) and transposing, formulas

(115) and (116) reduce to forms convenient for design as given in formulas (117) and (118). For vertical stirrups,

$$NA_v = \frac{\Sigma V'}{14{,}000d} \qquad (117)$$

For inclined stirrups,

$$NA_v = \frac{\Sigma V' \sin \alpha}{14{,}000d} \qquad (118)$$

in which A_v = right cross-sectional area of a single stirrup (both legs of a U-stirrup) and α = angle between the stirrup and the horizontal.

Table 67 gives all values of NA_v for 1 to 20 U-shaped stirrups of the usual sizes, for use with formulas (117) and (118). In using Table 67, the maximum permissible size of stirrup for any value of d should be determined from Diagram 66. If larger stirrups are used, the anchorage must be increased or the stress decreased. In any case the area $\Sigma V'$ (the vertically-hatched portion in Fig. 7 or Fig. 9) must be computed, taking the shears in pounds and distances along the beam axis in *inches*. In this computation the distance, a, along the axis, requiring web reinforcement must be determined. In Case I, a is equal to the distance from the face of the support to the load; or if a trapezoid between two loads were involved, a is equal to the base of the particular trapezoid under the shear curve for which stirrups are being designed. For Case II, with a triangular area under the shear curve, the value of a is found from formula (119).

$$a = \left(\frac{V - V_c}{V}\right)(c) \qquad (119)$$

in which c = the base length of the triangle in inches. For Case III in like manner (see Fig. 9 and Fig. 10) the value of a is given by formula (120).

$$a = \left(\frac{V'_1}{V_1 - V_2}\right)(c) \qquad (120)$$

Spacing of Stirrups.—Having calculated the number and size of stirrups required as shown above the spacing must be determined. Each stirrup should be so located as to take care of two equal unit trapezoids under the shear curve (see Fig. 8 in which a_1, a_2, a_3, etc., are the equal unit areas, each equal to $\Sigma V' \div 2N$) and should therefore be located at the junction line between these areas, for example between a_1, and a_2, a_3 and a_4, etc. The distances from the high end of the trapezoid under the shear curve to these points are designated x_1, x_3, x_5, etc., and are determined by formulas (121), (122), etc.:

$$\frac{x_1}{a} = \frac{V'_1}{V'_1 - V'_2} - \sqrt{\left(\frac{V'_1}{V'_1 - V'_2}\right)^2 - \left(\frac{V'_1 + V'_2}{V'_1 - V'_2}\right)\left(\frac{1}{2N}\right)} \qquad (121)$$

$$\frac{x_3}{a}=\frac{V'_1}{V'_1-V'_2}-\sqrt{\left(\frac{V'_1}{V'_1-V'_2}\right)^2-\left(\frac{V'_1+V'_2}{V'_1-V'_2}\right)\left(\frac{3}{2N}\right)}\quad\ldots.(122)$$

and so on, the number in the numerator of the final fraction in each formula corresponding to the subscript of x. Diagrams 69, 70 and 71 give values of $\frac{x_1}{a}$, $\frac{x_3}{a}$, $\frac{x_5}{a}$, etc., for all variations in the ratio of $\frac{V'_2}{V'_1}$ and for 1 to 20 stirrups. Having determined a, by formula (119) or (120) or directly from the shear diagram, the distances to the stirrup locations are taken directly from Diagrams 69, 70 and 71, using 72 as an aid, as explained in the instructions for their use.

With uniform load only on the beam, Case II, the values of V'_2 and of $\frac{V'_2}{V'_1}$ become zero and the values of $\frac{x_1}{a}$, $\frac{x_3}{a}$, etc., appear on the lower line of each section of Diagrams 69, 70 and 71. These same values are arranged in Table 68 with spacing grouped in the usual practical manner, and afford an especially rapid but accurate computation of stirrup spacing for uniform load.

Total Number of Stirrups.—The number of stirrups, N, found from the diagrams is the theoretical minimum number necessary to provide the required resistance, $\Sigma V'$. In the final design account must be taken of the limitations imposed by Section 804 of the Code. If the theoretical spacing is written down as it is read from the diagram a casual inspection will show how many stirrups must be added to comply with the rules governing maximum stirrup spacing. Problem 7 shows the complete solution of a general case, including the consideration of extra stirrups to meet these rules, while Problem 1 shows a solution using Table 68.

More Than 20 Stirrups Required.—For this case the designer will generally resort to a UU-shaped stirrup rather than to use too close a spacing. More than twenty stirrups to a single trapezoid under the shear curve is ordinarily undesirable. With UU-stirrups the values in Table 67 will be doubled. If, however, the designer desires to use more than 20 stirrups he may proceed as indicated in Fig. 11, dividing the original trapezoid into two smaller ones and making two solutions. If the lower trapezoid represents the shear value of 20 U-stirrups, the spacing in the higher of the two trapezoids may be considered as uniform and equal to the value by formula (123).

$$s=\frac{14{,}000\,A_v\,dx}{\Sigma V'}\quad\ldots\ldots\ldots\ldots(123)$$

in which $\Sigma V'$ is the area of the higher of the two trapezoids. The end

spacing will be $\frac{s}{2}$. The division point in Fig. 11 will be at a distance, x, from the high side of the original trapezoid determined by formula (124).

$$\frac{x}{a'} = \frac{V'_1}{V'_1 - V'_2} - \sqrt{\left(\frac{V'_1}{V'_1 - V'_2}\right)^2 - \left(\frac{V'_1 + V'_2}{V'_1 - V'_2}\right)\left(\frac{N' - 20}{N'}\right)} \quad \ldots (124)$$

Bent-up Bars as Web Reinforcement.—Fig. 12 illustrates the common case of a bar bent up in crossing from the bottom of the beam to the top of the beam. Under the Joint Code such a bar may be considered as effective web reinforcement over the center three-quarters of its sloping portion and to have a value over this distance, a_2, determined from formula (125).

$$V' = 16{,}000 A_v \sin a \quad \ldots\ldots\ldots\ldots (125)$$

In Fig. 12 the unshaded area represents the portion of $\Sigma V'$ taken by this

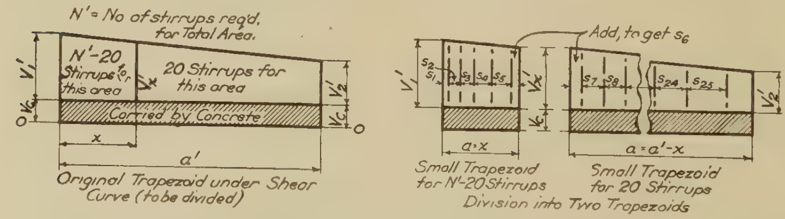


FIG. 11.—METHOD OF APPLYING SPACING DIAGRAMS WHEN MORE THAN TWENTY STIRRUPS ARE USED.

bar, leaving the two trapezoids, 1 and 2, and the triangle, 3, to be reinforced by stirrups. Values of V' for bars bent up at various slopes are given in Diagram 73. Problem 7 shows the complete design of a beam in which such a bent-up bar provides a portion of the web reinforcement at one end.

PROBLEM 7.

For purposes of comparison design the web reinforcement at one end of the beam shown by Figure 13, (a) using vertical stirrups and (b) using bent-up beam bars in conjunction with stirrups. Assume 2,000-lb. concrete in each case.

Solution: The beam is 12 by 36 in. For fire resistive construction $d = 36 - (1\frac{1}{2} + \frac{1}{2} + \frac{5}{8}) = 33.4$ in.

By formula (114) $V_c = (0.02)(2{,}000)(\frac{7}{8})(12)(33.4) = 14{,}000$ lb.

Laying this off from the shear axis as shown in Fig. 13 the value of V'_1 is 15,000 and of $V'_2 = 12{,}150$ and the ratio is:

$$\frac{V'_2}{V'_1} = \frac{12{,}150}{15{,}000} = 0.81$$

This is Case I and the value of a is 76 in. by inspection.

(a) *Design Using Vertical Stirrups.*—The area under the shear curve to be carried by the vertical stirrups, shown vertically-hatched in the diagram is:

$$\Sigma V' = \left(\frac{12{,}150 + 15{,}000}{2}\right)(76) = 1{,}030{,}000 \text{ in. lb.}$$

From Diagram 66 the maximum size of vertical stirrup for $d = 33.4$ and 2,000 lb. concrete is ⅜ in. rd., if plain, or ½ in. rd., if deformed. Use

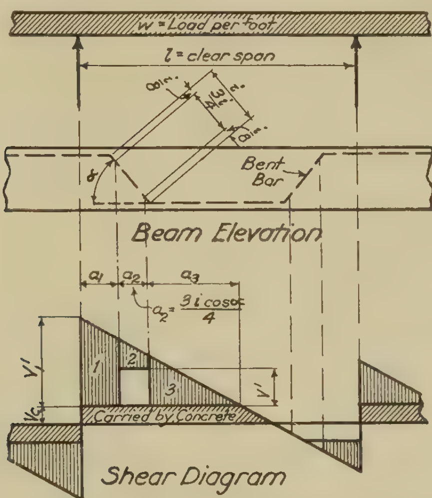


FIG. 12.—BENT-UP BARS IN CONJUNCTION WITH STIRRUPS.

½ in. rd. deformed bar for stirrups. By formula (117)

$$NA_v = \frac{1{,}030{,}000}{(14{,}000)(33.4)} = 2.21 \text{ sq. in.}$$

From Table 67, 6-½ in. rd. U-stirrups provide 2.36 sq. in. Enter Diagram 69, using the second section from the bottom (for 6 stirrups); the distance from the face of the support for $V'_2 \div V'_1 = 0.81$ are:

1st	stirrup $= 0.08a = (0.08)(76) =$	6 in.	Stirrup spacing 6 in.
2nd	stirrup $= .24a = (0.24)(76) =$	18 in.	Stirrup spacing 12 in.
3rd	stirrup $= .39a = (0.39)(76) =$	30 in.	Stirrup spacing 12 in.
4th	stirrup $= .56a = (0.56)(76) =$	43 in.	Stirrup spacing 13 in.

5th stirrup = .74a = (0.74) (76) = 56 in. Stirrup spacing 13 in.
6th stirrup = .91a = (0.91) (76) = 69 in. Stirrup spacing 13 in.

Since $v = \frac{(8)(29{,}000)}{(7)(12)(23.4)} = 83$ lb. per sq. in. ($= 0.42f'_c$) the maximum spacing by section 804 of the code is (¾) (33.4) = 25 in. and the design is satisfactory, calling for 6-½ in. rd. U-stirrups spaced 6 in., 2 at 12 in., 3 at 13 in., starting at the face of the support at each end of the span.

(b) *Design Using Bars Bent-up in Single Plane.*—By section 805c of

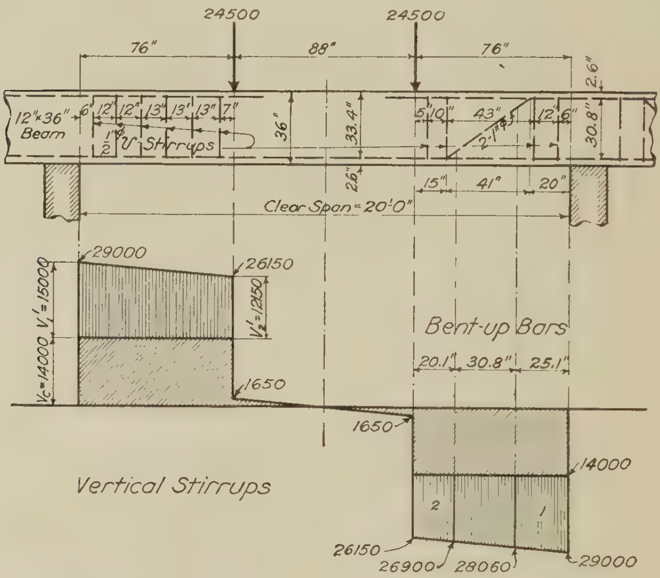


FIG. 13.—STIRRUPS VS. BENT-UP BARS (PROBLEM 7).

the code, only the center three-quarters of the sloping portion may be considered effective as web reinforcement.

It will generally prove wasteful to attempt to bend the bar so that the effective zone will cover the entire 76 in. that requires web reinforcement. The points of bend would have to lie (1/6) (76) = 12.7 in. inside the face of the support and beyond the load point, so that the bar would be of very little value as tensile reinforcement at sections of maximum negative and positive moment. It will be better to add stirrups at either end of the trapezoid and make the upper point of bend 20 in. outside the face of the support and the lower point of bend 15 in. inside the load point (see Fig. 13) where the moments are such that this steel may be bent down

without requiring extra tensile reinforcement. The length of the "run" of the bar (Diagram 73) will be $76 - (15 + 20) = 41$ in. The "rise" will be $36 - [(2)(2.6)] = 30.8$. Ratio of "run" to "rise" $= \frac{41}{30.8} = 1.33$. From Diagram 73, lower part, the value of 1 — 1 in. rd. bar at this ratio is 7,500 lb. and 2 — 1 in. rd. bars will be required to carry $V'_1 = 15{,}000$ lb.

The zone, a_2, in which these bars are effective will end at $(\frac{1}{8})(41) = 5.1$ in., from the points of bending of the bar and will be $41 - [(2)(5.1)] = 30.8$ in. wide located as shown in Fig. 13. The 25.1 in. at the support will require 2-½ in. rd. U's while the 20.1 in. next to the load point will require 2-½ in. rd. U's, as is readily determined by the stirrup design already made. The stirrup spacing from the face of the support may be taken as 6 in., 12 in., 38 in., 13 in. from the previous stirrup design.

A computation of the stirrups required by the small trapezoid 2 will give a check on the results as follows:

$$\Sigma V' = \left(\frac{12{,}900 + 12{,}150}{2} \right) (20.1) = 252{,}000 \text{ in. lb.}$$

$$NA_v = \frac{252{,}000}{(14{,}000)(33.4)} = 0.54 \text{ sq. in.} = 2\text{-}\tfrac{1}{2} \text{ in. rd.}$$

$$\frac{V'_2}{V'_1} = \frac{12{,}150}{12{,}900} = 0.94 \qquad a = 20.1 \text{ in.}$$

From Diagram 69 the distance from edge of zone covered by bent-up bars to first stirrup $= (0.25)(20.1) = 5$ in.
To second stirrup $= (0.75)(20.1) = 15$ in.

Revised spacing of stirrups will be 6 in., 12 in., 43 in., 10 in.

Two-way Slabs Supported on Beams.—Diagram 74 gives the load distribution for design strips (commonly taken as 12 in. wide) in each direction in ordinary slabs supported on beams on all four edges. The design of the unit strips within the middle half of the clear span in each direction employs the same moment coefficients as are used for beams under the same general conditions of loading, support and restraint. In the outer quarters the reinforcement is permitted to be reduced to one-half of that required in the parallel middle strips. The supporting beam must be designed to carry in addition to its own weight and superimposed live load a uniform load throughout its length equal to the load per foot brought to it by the middle strips on either side. No reduction in live load is permitted on beams supporting two-way slabs, even though such reduction may be used under the code for beams supporting one-way slabs. The design of the strips is the same as the design of rectangular beams.

Quantities of Concrete, Formwork and Reinforcement.—In the design of beams and T-beams architectural considerations, such as unobstructed head room, arrangement of beams on ceiling, etc., are so important that

the range in beam sizes and proportions, even in buildings of the same span and floor load, is considerable. I have not attempted, therefore, to give any formulas or diagrams for quantities of materials required in the beam-and-slab types of structure. Part II of this paper, however, gives tables of quantities from actual designs for quite a wide range of panel sizes, live loads and concrete strengths. In the case of columns, likewise, there is ordinarily some architectural limitation on size that introduces wide diversity in designs for the same vertical load, and there is very commonly some bending moment present to modify the final design. It is therefore difficult in this case also to prepare any formulas or diagrams for quantities of materials that would be generally applicable. Where members can be standardized, as is the case with flat slab floors and with spread footings, it is not difficult to prepare diagrams giving quite closely the volume of concrete, area of formwork and weight of reinforcing steel required for typical design conditions.

For flat slab floors each *design* diagram is followed on the facing page by a *quantity* diagram for the same conditions. The quantities of concrete, formwork and reinforcement required for a design taken from Diagram 77, for example, may be read directly from Diagram 78. For square spread footings the same arrangement has been followed. The quantities for a design selected from Diagram 112, for example, may be taken directly from Diagram 113 on the facing page. In the case of footings this offers the additional advantage that the *excavation* required by the contractor may be taken from the design diagram (making proper allowances for the space occupied by the formwork) in the same operation without turning a page.

In using quantity diagrams allowance must always be made for extra concrete if a slab is made 8¼ inches thick where the design diagram calls for 8.2 inches, for example. The diagrams are right, but the actual design may vary slightly from the theoretical and a corresponding allowance must be made by the estimator. In the same way some allowance should always be made for extra steel in cases where the steel area from the diagram does not divide evenly into bars. The diagram may call for 6.8, ½-in. round bars, but the design will call for seven, and the estimator must add for this contingency. It is impossible to make such allowances in the diagrams since one designer may use ½-in. round bars while another may use ⅝-in. round bars, and the allowance would be different.

These quantity diagrams are useful to the designer as well as to the estimator. It enables the designer to study the relative economy for different concretes, varying spans and soils of unequal bearing power, with a great saving in time and labor. In using quantity diagrams for comparative purposes in design it is not necessary to make any allowances, as mentioned in the last paragraph, since these allowances will balance very closely and will not affect the comparisons materially.

Flat Slab of Standardized Proportions.—The Joint Code properly permits a wide latitude in the proportions of flat slab floors, to permit of columns without capitals, or without drops, or with unusually large or

small capitals, such as frequently are required. For the usual run of factory buildings, however, a standardized design is entirely acceptable and will save much labor. I have used a column capital diameter equal to 0.225 l in Diagrams 79, 83, etc., while in Diagrams 77, 81, etc., the capital size varies from 0.225 l to 0.25 l according to standard metal column form used. Diagrams 79, 83, etc., may be used with wood column capital forms where no standards govern but only Diagrams 77, 81, etc., should be used for circular capitals formed in metal molds in the usual way. The side of the square dropped panel is taken as 0.35 l in all cases. The depth of the dropped panel below the slab is taken as one-half the slab thickness in all cases. The following moments are taken which lie within the values given in table in Section 1003 of the code:

Two-way system	*Four-way system*
$-M_c = 0.47\ M_o;\ +M_c = 0.21\ M_o$	$-M_c = 0.51\ M_o;\ +M_c = 0.20\ M_o$
$-M_m = 0.16\ M_o;\ +M_m = 0.16\ M_o$	$-M_m = 0.09\ M_o;\ +M_m = 0.20\ M_o$

This distribution of the total bending moment gives somewhat different values for the square four-way panel than those stated in Section 1004c of the code, which is merely one of several distributions permitted. The distribution used in the diagrams and tables of this paper results in a simpler design for the standardized proportions. Designs made in accordance with this office standard may be taken direct from Diagrams 77, 79, etc., in accordance with the instructions under Table 75, for all cases of square interior flat slab floor panels surrounded by other panels of approximately the same size. Problem 8 shows a complete design for such a panel for both two-way and four-way systems. The length of bars in the various bands must be determined from the provisions of Sections 1007 to 1010 of the Joint Code.

Where exterior panels in flat slab floors are of the same size and shape as the adjoining interior panels and have regular column capitals, the column strip or direct band lying partly in the interior panel and partly in the exterior panel will be the same as for an interior panel by Table 75. The middle strip of a two-way system parallel to the wall will be the same as for an interior panel by Table 75. The column strip or direct band along the wall will be (proportional to its width) the same as a similar interior strip or band except as affected by the provisions of Section 1012 of the code. The top band across the direct band, and extending from an interior to an exterior panel, and also extending between two exterior panels will be the same as for an interior panel by Table 75. The remaining design strips or bands will take the reinforcement called for by Table 76. The slab thickness, drop thickness, etc., will be governed by Table 75 and Diagrams 77, 79, 81, etc., to 91.

For rectangular, irregular or special panels, the tables and diagrams are not applicable and the usual complete design process must be resorted to. Even for such cases, however, the diagrams afford an excellent basis

for judgment in assuming slab thicknesses, etc., or in making rough estimates.

PROBLEM 8.

Design the typical square interior and exterior panels of a flat slab floor (*a*) with four-way reinforcement and (*b*) with two-way reinforcement, assuming 2,000-lb. concrete. The wall columns have a half regular column capital. Use a live load of 200 lb. per sq. ft. and the Joint Code, with standard steel capital forms. All panels are 21 ft. by 21 ft. c. to c. of columns.

(a) *Design with Four-way Reinforcement.*—Solution: Diagram 77 in conjunction with Tables 75 and 76 give the design as follows:

From Diagram 77 for a 21 ft. square panel:

Side of square dropped panel = 7 ft. 4 in.
Slab thickness for 200-lb. LL = 8⅜ in.
The column capital will be 5 ft. in diameter.
Basic steel area for 200-lb. LL = 1.58 sq. in.

From Table 75:

Dropped panel thickness = (½) (8⅜) = 4¼ in.
This makes the dropped panel, 7 ft. 4 in. by 7 ft. 4 in. by 4¼ in.
Each top band, A_s = 1.58 sq. in. = 8-½ in. rd. bars.

For the typical interior panel:

Diagonal band—bent bars, A_s = (0.67) (1.58) = 1.05 sq. in. = 6-½ in. rd.

Diagonal band—straight bars, A_s = 1.58 sq. in. = 8-½ in. rd.
Direct band—bent bars, A_s = 1.58 sq. in. = 8-½ in. rd.
Direct band—straight bars,
A_s = (1.22) (1.58) = 1.93 sq. in. = 10-½ in. rd.

From Table 76, for the typical exterior panel:

Top band at and perpendicular to wall,
A_s = (0.625) (1.58) = 0.99 sq. in. = 5-½ in. rd.
Diagonal band—bent bars,
A_s = (1.13) (1.58) = 1.79 sq. in. = 9-½ in. rd.
Diagonal band—straight bars,
A_s = (0.75) (1.58) = 1.18 sq. in. = 6-½ in. rd.
Diagonal band—top bars over exterior col. head,
A_s = (0.22) (1.58) = 0.35 sq. in. = 2-½ in. rd.
Direct band, perpendicular to wall,—bent bars,
A_s = (1.67) (1.58) = 2.64 sq. in. = 14-½ in. rd.
Direct band, perpendicular to wall,—straight bars,
A_s = (1.11) (1.58) = 1.75 sq. in. = 9-½ in. rd.

The direct band, parallel to the wall, lying partly in the interior and partly in the exterior panel will be the same as the direct band of a typical interior panel.

The direct band (or half band, in most cases) lying along the wall will be designed in accordance with Section 1012 of the code.

(b) *Design With Two-way Reinforcement.*—The slab thickness, the dimensions of the square dropped panel and the column capital will be the same as for the four-way design above. The basic steel area will be 1.58 sq. in. also.

For the typical interior panel, from Table 75:

Middle strip—bent bars,

$A_s = (0.89)(1.58) = 1.41$ sq. in. $= 7$-½ in. rd.

Middle strip—straight bars,

$A_s = 1.58$ sq. in. $= 8$-½ in. rd.

Column strip—bent bars,

$A_s = (1.56)(1.58) = 2.46$ sq. in. $= 13$-½ in. rd.

Column strip—straight bars,

$A_s = (0.78)(1.58) = 1.23$ sq. in. $= 6$-½ in. rd.

Column strip—added over col. head,

$A_s = (0.36)(1.58) = 0.57$ sq. in. $= 3$-½ in. rd.

For the typical exterior panel, from Table 76:

Middle strip, perpendicular to wall, bent bars,

$A_s = (1.11)(1.58) = 1.76$ sq. in. $= 9$-½ in. rd.

Middle strip, perpendicular to wall, straight bars,

$A_s = (1.12)(1.58) = 1.77$ sq. in. $= 9$-½ in. rd.

Column strip, perpendicular to wall, bent bars,

$A_s = (1.95)(1.58) = 3.08$ sq. in. $= 16$-½ in. rd.

Column strip, perpendicular to wall, straight bars,

$A_s = (0.98)(1.58) = 1.54$ sq. in. $= 8$-½ in. rd.

Column strip, perpendicular to wall, bars in top over exterior column head,

$A_s = (1.18)(1.58) = 1.86$ sq. in. $= 9$-½ in. rd.

The intermediate strip parallel to the wall will have the same bars as an intermediate strip in an interior panel.

The column strip lying partly in the interior and partly in the exterior panel will be the same as the column strip of an interior panel.

The column strip lying along the wall will be designed in accordance with Section 1012 of the code.

(c) *Length of Bars.*—For either four-way or two-way reinforcement the points of bending, etc., and the points of termination of bars will be as given in Sections 1007 to 1010 of the code.

Long Columns.—The upper portion of Diagram 93 gives the ratio of the radius of gyration of a circular core with not over 6 per cent vertical reinforcement to the core diameter. The lower portion of Diagram 93 gives the proportionate load-carrying capacity of columns in which h/R exceeds 40 or 50 in accordance with formula (26) or (26a) of the code. The upper portion of Diagram 93 is based on the approximation that the effective diameter for the ring of longitudinal bars will be 0.9 of the core diameter. This will not apply to columns having the bars arranged in two rings or

to very small or very large columns. Such cases require that the value of R be computed.

Spiral Columns.—In spiral column design, having assumed a core diameter, the area of the spiral core is taken from Table 105 and the weight of the corresponding column is added to the applied load to secure the total design load on the column. Table 106 gives the volumes of square and round column shafts and column capitals. Compute the value of P/A and enter Diagram 94, or 95 to 98, according to the strength of concrete used in the design. Lay the edge of a triangle from the value of P/A on the left scale to the core diameter on the right scale and read off the spiral rod and pitch on the left center scale and the number and size of verticals on the right center scale, completing the design in a single operation. The percentage of spiral in Diagrams 94 to 98 is always one-fourth of the percentage of verticals as required in the code. Problem 9 shows the complete design of a spiral column by this brief method. The provisions of Section 1103 of the Joint Code regarding minimum requirements must be met.

PROBLEM 9.

Design an axially loaded reinforced-concrete column for a load of 1,100,000 lb., including assumed column weight, using 3,000-lb. concrete and the spiral type of column reinforcement, in accordance with the Joint Code. The unsupported length is 11 feet.

Solution: Assume 36-in. round column with a 32-in. diameter core section. The core area from Table 105 is 804.2 sq. in.

$$P/A = \frac{1,100,000}{804.2} = 1,368 \text{ lb. per sq. in.}$$

From Diagram 96, using a straight edge set to 1,368 on the left scale and 32 on the right scale, we read on the center scales

Vertical bars = 18 — 1¼ in. sq.
Spiral = ½ in. rd. at 2¾ in. pitch.

The ratio of unsupported length to core diameter $\left(\frac{132}{36} = 3.7\right)$ is so small as not to require consideration in connection with Section 1108 of the code.

Tied Columns.—Diagram 100 is the usual design diagram for tied columns, using the Joint Code. In the usual design procedure, a column size is assumed, the area of which may be taken from Table 107 for ordinary round columns or cylindrical piers, the weight of the corresponding column is added to the applied load and from the value of P/A the proper percentage of vertical reinforcement is read from Diagram 100 for the concrete strength used in the design. The area of vertical rods required will be p times the column area and the corresponding bars may be computed from Table 2 with code limitations observed. The design is completed by the selection of ties to meet the code requirements, Section 1104. Problem 10 shows the complete design of a tied column.

PROBLEM 10.

Design an axially loaded reinforced-concrete column for a load of 300,000 lb., including assumed column weight, using 2,000-lb. concrete and tied longitudinal bars, in accordance with the Joint Code. The unsupported length is 9 feet.

Solution: Assume a column 24-in. square. The area will be (24) (24) $= 576$ sq. in. and the unit load will be $P/A = \dfrac{300{,}000}{576} = 520$ lb. per sq. in.

From Diagram 100 for $P/A = 520$ and 2,000-lb. concrete the value of p is 0.011.

$A_s = (0.011)(576) = 6.34$ sq. in. $= 8 - 1$ in. rd. bars.

From Section 1104 of the code, the ties will be ¼ in. rd. spaced 12 in. o. c. and so arranged as to provide ties in two directions for the four bars at the middle of the sides of the column as well as for the four bars at the corners. The ties will be bent so as to keep the longitudinal bars 2 in. in the clear from the column surface at all points.

Composite and Combination Columns.—The design of composite and combination columns in accordance with Sections 1106 and 1107 of the code would require four additional diagrams. Space limitations of this paper indicate that diagrams so infrequently used should be omitted. The plotting data for these diagrams are as follows:

Value of p	Composite Columns								Combination Columns	
	Unit Stress on Reinforced Concrete Section with 1% Spiral Reinforcement and $f'c$ =					Unit Stress on Metal Core under Construction Loads			$\frac{A_s}{A}$	Increase in Stress
	2000	2500	3000	3750	5000	h/R	Structural Steel	Cast Iron		
0.02.......	640	763	886	1067	1375	60	15000	8400	0.1	0.090
0.025......	675	798	920	1100	1437	80	13280	7200	0.2	0.040
0.03.......	710	832	954	1133	1500	100	11580	6000	0.3	0.023
0.035......	745	866	987	1166	1562	120	10000	4800	0.4	0.015
0.04.......	780	900	1021	1200	1625	140	8620	3600	0.5	0.010

Spread Footings.—The Joint Code properly leaves the designer considerable latitude in the design of concrete footings resting directly on the soil. A standardization of proportions, putting all horizontal dimensions in terms of b, the dimension of the side of a square footing, and all vertical dimensions in terms of d, the effective depth, greatly simplifies the formulas and expedites the design. Fig. 16 shows the standard proportions adopted for flat-top footings and Fig. 17 those for sloping-top footings. With these constant ratios, the various design requirements can all be expressed in terms of w, the soil load (neglecting the footing weight which is carried

directly to the soil without shear or moment, but which must be used in computing the area of the footing), b and d. The bending moment at a section in the plane of the face of the pier is found by formula (126).

$$M = 0.6Pb \quad \ldots\ldots (126)$$

in which

M is in inch-pounds

P is the load in pounds at the top of the footing

b is the side of the base of the square footing in feet.

For *flat-top* square footings the value of d/b is found by solving formula (127) (answer read directly from Diagram 108).

$$\frac{w}{v_c} = \frac{126\left(\frac{d}{b}\right)\left(8\frac{d}{b}+1\right)}{1-\left(2\frac{d}{b}+.25\right)^2} \quad \ldots\ldots (127)$$

For *sloping-top* square footings the value of d/b is found by solving formula (128) (answer read directly from Diagram 108).

$$\frac{w}{v_c} = \frac{270\left(\frac{d}{b}\right)\left(8\frac{d}{b}+1\right)\left(.491-\frac{d}{b}\right)}{1-\left(2\frac{d}{b}+.25\right)^2} \quad \ldots\ldots (128)$$

In these formulas v_c may be taken at $0.02f'_c$ or $0.03f'_c$ depending on the anchorage of the footing reinforcement. Diagrams 113, 114, etc., which follow are all based on $v_c = 0.03f'_c$ with all footing bars hooked, but Diagram 108 is perfectly general and applies to any value of v_c.

When using the standard proportions of Fig. 16 or Fig. 17 it is not necessary to compute the bending moment, since the proportions are selected to give concrete stresses at or just under the permitted values. The area of steel in *each* of two directions may be computed directly from formula (129).

$$A_s = \frac{0.05P}{17{,}500\left(\frac{d}{b}\right)} \quad \ldots\ldots (129)$$

The value of d/b is known from Diagram 108. To complete the design it is only necessary to select the number and size of bars to make up the required A_s, safeguarding the bond stress by not exceeding the size of bar indicated by Diagram 109. If the bar size, D, is taken as not larger than the value by formula (130) the bond stress will be satisfactory.

$$D = 0.0000427\ bu \quad \ldots\ldots (130)$$

For deformed bars with hooked ends as specified in Section 903 of the code formula (130) reduces to $D = 0.0064b$ for 2,000-lb. concrete, and to $D = 0.0096b$ for 3,000-lb. concrete. Diagram 109 gives the maximum allowable bar size for any given value of b by formula (130). These formulas

and the values from Diagram 109 apply to both flat-top and sloping-top footings. In flat-top footings the bond unit stress decreases rapidly at sections away from the face of the pier. In sloping-top footings of the proportions used here, the bond unit stress remains almost constant to a point on and near the top of the slope and then decreases to the edge. Hooked bars are required.

In making a design for a fixed soil pressure which includes the weight of the footing it is necessary to determine the weight per sq. ft. of the footing. For *flat-top* footings this is given by formula (131).

$$w' = 12.5d + 50 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (131)$$

For *sloping-top* footings it is given by formula (132).

$$w' = 7.5d + 50 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (132)$$

In (131) and (132) the term 50 is the weight of the four inches of concrete beneath the centroid of the reinforcing steel.

The volume of concrete in sloping-top footings will be given very closely by formula (133).

$$\text{Volume} = (0.05d + 0.333)\,b^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (133)$$

where d is in inches, b is in feet and the *volume* is in cu. ft.

The minimum edge thickness of sloped-top footings is ten inches by the code. If d is less than 24 in. (28 in. total thickness) the weight of footing and volume of concrete in sloped-top footings will be increased. The quantity diagrams include this increase, but formulas (132) and (133) require adjustment for this case.

The depth of a footing of any fixed proportions depends primarily on the superimposed load and is almost independent of the soil pressure used. Diagram 111 gives the depth of footings of the types shown in Fig. 16 and Fig. 17 for 6,000-lb. soil and the depth for any other usual soil pressure will be within a few per cent of these values for the same load. In general other soil pressures will give slightly less depths. 6,000-lb. soil appears to give the maximum depth within the range of soil pressure usually used under spread footings. Selections from Diagram 111 will not need to be checked for diagonal tension or shear, except to determine a possible slight saving in depth.

Diagrams 112, 114 to 126 are design graphs for sloping-top footings for soil pressures of 3,000, 4,000, 5,000 and 6,000-lb. per sq. ft. Each diagram gives values for two strengths of concrete. Similarly, Diagrams 128, 130 to 142 cover the design for flat-top footings for the same soil pressures and concrete strengths. These diagrams *allow accurately* for the footing weights and give the indicated soil pressure when subjected to the axial load shown in the diagrams. Enter the diagrams at the top with the total applied load at the top of the footing (the basement column load plus the assumed weight of the pier) and drop vertically reading off the value of b, of t (the total footing depth) and of A_s, in succession. Complete the design by computing the other dimensions of the footings from b and d, in accordance with Fig. 16 or Fig. 17 and by selecting the number and size of bars to make up the required A_s, using bars not larger than

indicated by Diagram 109. Problem 11 shows the complete design of a square footing of the flat-top type for the same basement story column load used in Problem 9. In Problem 12 the design of the top of the pier for the safe transfer of the load without exceeding Joint Code stresses is shown, using Diagram 110 for a rapid solution of formula (109) of the code.

Quantity Diagrams for Footings.—Facing each design diagram, a diagram giving the quantities of concrete, formwork and reinforcing steel for single footings is given. The basis and use of these diagrams is described more fully on page 566, which should be consulted.

PROBLEM 11.

Design a flat-top square spread footing for the column given in Problem 9, using a 3,000-lb. concrete in the footing and designing for a 4,000-lb. per sq. ft. soil pressure.

Solution: Enter Diagram 130 at the top with the load of 1,100,000 lb. Drop vertically and read off the dimension of the side of the square footing at the upper index line for 3,000-lb. concrete as 17 ft. 6 in. Continue vertically down to the middle index line for 3,000-lb. concrete and read off the depth as 29½ in. The footing will be 17 ft. 6 in. by 17 ft. 6 in. by 2 ft. 6 in.

Continue vertically downward to the lower index line for 3,000-lb. concrete and read off the steel area as 26.0 sq. in. in each of two directions. From Diagram 109 the maximum bar size for a 17 ft. 6 in. footing using 3,000-lb. concrete is not limited. Use 21 — 1⅛ in. sq. bars in each direction. The length of each bar including the hooks at each end will be:

(17 ft. 6 in.) + [(20) (1⅛)] = 19 ft. 5 in. as a minimum.

From Fig. 17 the side of the square pier will be

(0.25) (17 ft. 6 in.) = 4 ft. 4½ in.

The height of the pier, for a 36-in. column as designed in Problem 10, will be:

(4 ft. 4½ in.) — (3 ft. 0 in.) = 1 ft. 4½ in. (minimum).

PROBLEM 12.

Design the top of the pier in Problem 11 for the transfer of the load from the column of Problem 10 to the pier, in accordance with the Joint Code.

Solution: Section 1205 of the code applies:

A = Area of top of pier = (52.5) (52.5) = 2,756 sq. in.

A' = Area of 32-in. round column = 804 sq. in.

$$A/A' = \frac{2756}{804} = 3.43$$

$$r_a \text{ at the base of the column } = \frac{1{,}100{,}000}{804} = 1368.$$

Enter Diagram 110, upper portion, with $A/A' = 3.43$ and move across to $r_a = 1{,}368$. 3,750-lb. concrete is required, without spiral reinforcement, in the pier.

TABLE 1.—MINIMUM BEAM WIDTHS (IN INCHES)

Size of Bars	Number of Bars in Single Layer of Reinforcing							Add for each Added Bar
	2	3	4	5	6	7	8	
A. ORDINARY CONSTRUCTION—END OF BARS NOT SPECIALLY ANCHORED								
1/2 in. round	4	5 1/2	7	....	....	....	....	1 1/2
1/2 in. square	4 1/4	6	7 3/4	....	....	....	....	1 3/4
5/8 in. round	4 1/4	5 7/8	7 1/2	9 1/8	10 3/4	....	....	1 5/8
3/4 in. round	4 5/8	6 1/2	8 3/8	10 1/4	12 1/8	....	....	1 7/8
7/8 in. round	5 1/16	7 1/4	9 7/16	11 5/8	13 13/16	16	18 3/16	2 3/16
1 in. round	5 1/2	8	10 1/2	13	15 1/2	18	20 1/2	2 1/2
1 in. square	6	9	12	15	18	21	24	3
1 1/8 in. square	6 1/2	9 7/8	13 1/4	16 5/8	20	23 3/8	26 3/4	3 3/8
1 1/4 in. square	7	10 3/4	14 1/2	18 1/4	22	25 3/4	29 1/2	3 3/4
B. ORDINARY CONSTRUCTION—END OF BARS SPECIALLY ANCHORED								
1/2 in. round	4	5 1/2	7	....	....	....	....	1 1/2
1/2 in. square	4 1/4	6	7 3/4	....	....	....	....	1 3/4
3/8 in. round	4 1/4	5 7/8	7 1/2	9 1/8	10 3/4	....	....	1 5/8
3/4 in round	4 1/2	6 1/4	8	9 3/4	11 1/2	....	....	1 3/4
7/8 in. round	4 3/4	6 5/8	8 1/2	10 3/8	12 1/4	14 1/8	16	1 7/8
1 in. round	5	7	9	11	13	15	17	2
1 in. square	5 1/2	8	10 1/2	13	15 1/2	18	20 1/2	2 1/2
1 1/8 in. square	5 15/16	8 7/8	11 9/16	14 3/8	17 3/16	20	22 13/16	2 13/16
1 1/4 in. square	6 3/8	9 1/2	12 5/8	15 3/4	18 7/8	22	25 1/8	3 1/8

INSTRUCTIONS FOR USE.—This table shows the minimum width of beam stem, without stirrups, in which the longitudinal bars are covered by one inch of concrete and are spaced as follows:

For round bars not specially anchored, 2 1/2 bar diameters between centers.

For square bars not specially anchored, 3 times the side dimension between centers.

For round bars with special anchorage, 2 bar diameters between centers.

For square bars with special anchorage, 2 1/2 times the side dimensions between centers.

To the widths shown in the table additions must be made as follows:

1. Add the width of any stirrup legs placed between the longitudinal bars and the sides of the beam.
2. Add 1 in. for fire resistive construction or 2 in. for special exposure or spalling aggregates.
3. Add extra width as required, in case aggregate exceeds 3/4 in. in size, to insure a clear space between bars of not less than 1 1/3 times the maximum size of the coarse aggregate.

For more bars of any size than covered by the table or for bars of a different size, add the dimension given in the last column for each such extra bar. (See 1928 Joint Code, Sections 504, 506, and 903)

TABLE 2.—AREAS, PERIMETERS AND WEIGHTS OF PLAIN BARS

Number of Bars		Sizes of Plain Bars										
		¼ in. round	⅜ in. round	½ in. round	½ in. square	⅝ in. round	¾ in. round	⅞ in. round	1 in. round	1 in. square	1⅛ in. square	1¼ in. square
1.......	A	0.0491	0.1104	0.1963	0.250	0.3068	0.4418	0.6013	0.7854	1.00	1.2656	1.5625
	o	0.785	1.178	1.571	2.000	1.964	2.356	2.749	3.142	4.00	4.500	5.000
	W	0.167	0.375	0.668	0.850	1.043	1.502	2.044	2.670	3.40	4.303	5.313
2.......	ΣA	0.10	0.22	0.39	0.50	0.61	0.88	1.20	1.57	2.00	2.53	3.12
	Σo	1.57	2.36	3.14	4.00	3.93	4.71	5.50	6.28	8.00	9.00	10.00
	ΣW	0.33	0.75	1.34	1.70	2.09	3.00	4.09	5.34	6.80	8.61	10.63
3.......	ΣA	0.15	0.33	0.59	0.75	0.92	1.33	1.80	2.36	3.00	3.80	4.69
	Σo	2.36	3.53	4.71	6.00	5.89	7.07	8.25	9.42	12.00	13.50	15.00
	ΣW	0.50	1.12	2.00	2.55	3.13	4.51	6.13	8.01	10.20	12.91	15.94
4.......	ΣA	0.20	0.44	0.78	1.00	1.23	1.77	2.41	3.14	4.00	5.06	6.25
	Σo	3.14	4.71	6.28	8.00	7.86	9.42	11.00	12.57	16.00	18.00	20.00
	ΣW	0.67	1.50	2.67	3.40	4.17	6.01	8.18	10.68	13.60	17.21	21.25
Area of 12 Bars........		0.59	1.32	2.36	3.00	3.68	5.30	7.22	9.41	12.00	15.18	18.75

INSTRUCTIONS FOR USE.—This table is based on the properties of plain round or plain square and may be used in design for areas and perimeters of deformed bars. For weights of deformed bars manufacturers' tables should be consulted.

Cross sectional areas (A or ΣA) of bars are given in sq. in.

Perimeters (o or Σo) are given in inches.

Weights (W or ΣW) are given in lb. per foot of length.

The last line gives values of $12a_S$ for use in the formula:

$$\text{Spacing of bars in slab (in inches)} = \frac{12a_S}{A_S}.$$

in which $12a_S$ = cross sectional area of 12 slab bars of size used.

A_S = cross sectional area per foot width of slab required.

Areas, perimeters and weights of groups of more than four bars may be easily obtained from the table by simple additions or multiplications.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 3.—RECTANGULAR BEAMS. $f_c = 0.40\,f'_c$

2000-lb. Concrete	2500-lb. Concrete	3000-lb. Concrete	3750-lb. Concrete
$n = 15$ $f_c = 800$	$n = 12$ $f_c = 1000$	$n = 10$ $f_c = 1200$	$n = 8$ $f_c = 1500$
$p = 0.0075$ $K = 131$	$p = 0.0094$ $K = 164$	$p = 0.0112$ $K = 197$	$p = 0.0140$ $K = 246$

$k = 0.375$ for all of the above cases.

INSTRUCTIONS FOR USE.—The value of f_c is 0.4 f'_c and is the compressive stress to be used in flexure calculations, except for the special case covered by Table 4. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 4.—RECTANGULAR BEAMS. $f_c = 0.45 f'_c$

2000-lb. Concrete	2500-lb. Concrete	3000-lb. Concrete	3750-lb. Concrete
$n = 15$ $f_c = 900$	$n = 12$ $f_c = 1125$	$n = 10$ $f_c = 1350$	$n = 8$ $f_c = 1688$
$p = 0.0091$ $K = 157$	$p = 0.0113$ $K = 196$	$p = 0.0136$ $K = 235$	$p = 0.0170$ $K = 294$

$k = 0.403$ for all of the above cases.

INSTRUCTIONS FOR USE.—The value of f_c is $0.45\,f'_c$ and its use is limited to designs for flexure adjacent to supports of continuous or fixed beams or of rigid frames. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 5.—T-BEAMS. $f_c = 0.40\,f'_c$

$\frac{t}{d}$	2000-lb. Concrete		2500-lb. Concrete		3000-lb. Concrete		3750-lb. Concrete	
	$n = 15$, $f_c = 800$		$n = 12$, $f_c = 1000$		$n = 10$, $f_c = 1200$		$n = 8$, $f_c = 1500$	
	p	K	p	K	p	K	p	K
0.04	0.0015	30	0.0019	37	0.0023	45	0.0028	56
0.06	0.0022	43	0.0028	54	0.0033	64	0.0041	80
0.08	0.0029	55	0.0036	69	0.0043	82	0.0054	103
0.10	0.0035	66	0.0043	83	0.0052	99	0.0065	124
0.12	0.0040	76	0.0050	95	0.0061	114	0.0076	143
0.14	0.0046	85	0.0057	107	0.0068	128	0.0085	160
0.16	0.0050	93	0.0063	117	0.0076	140	0.0094	175
0.18	0.0055	101	0.0068	126	0.0082	151	0.0103	189
0.20	0.0059	107	0.0073	134	0.0088	161	0.0110	201
0.22	0.0062	113	0.0078	141	0.0093	169	0.0117	211
0.24	0.0065	117	0.0082	147	0.0098	176	0.0122	220
0.26	0.0068	121	0.0085	152	0.0102	182	0.0127	228
0.28	0.0070	125	0.0088	156	0.0105	187	0.0132	234
0.30	0.0072	128	0.0090	160	0.0108	192	0.0135	240
0.32	0.0073	129	0.0092	161	0.0110	194	0.0138	242
0.34	0.0074	130	0.0093	163	0.0112	196	0.0139	245
0.36	0.0075	131	0.0094	164	0.0112	197	0.0140	246
$k = 0.375$	0.0075	131	0.0094	164	0.0113	197	0.0140	246

$k = 0.375$ for all of the above cases.

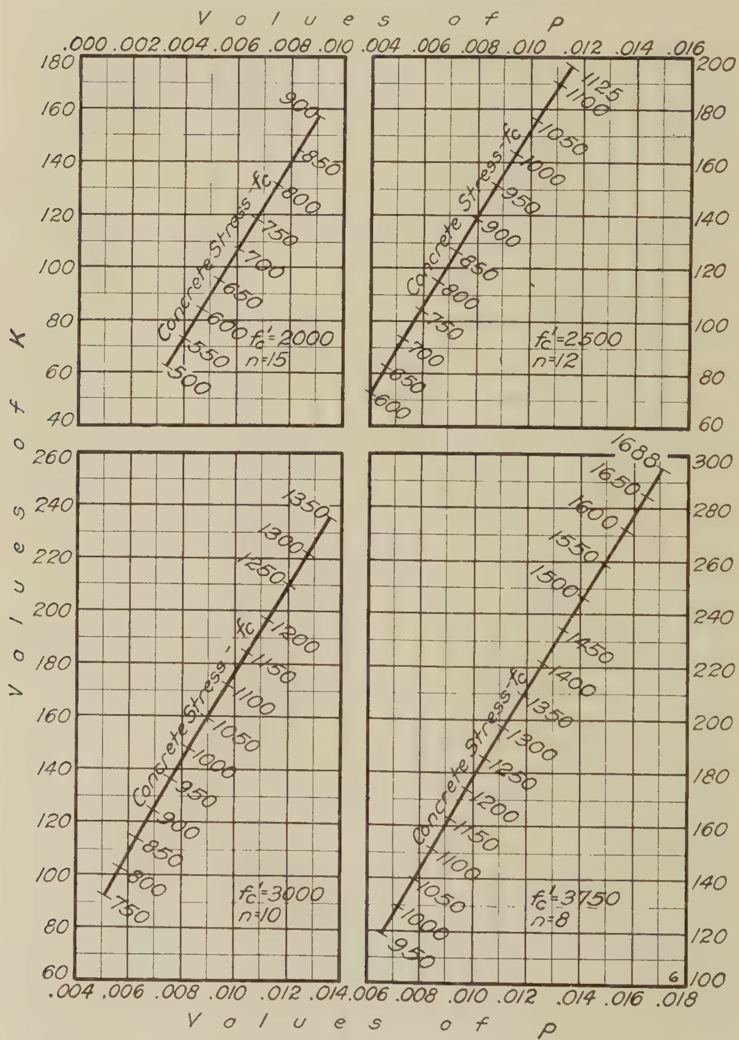
INSTRUCTIONS FOR USE.—The compressive stress in the stem of the beam, between the neutral axis and the lower face of the flange is not made use of in the table. Where many T-beams are involved having small values of t/d and wide stems, this additional strength may be taken in account in design.

GENERAL NOTE FOR TABLES 3–19 INCLUSIVE

The resisting moment of a beam with balanced reinforcement is equal to Kbd^2. The area of the tensile reinforcement is equal to pbd. The area of the compressive reinforcement, where used, is equal to $p'bd$. A beam with balanced reinforcement is one in which the full tensile and compressive unit stresses shown in the tables are developed. For steps to be taken in design of beams consult text.

DESIGN VALUES OF p AND K FOR $f_s = 20,000$

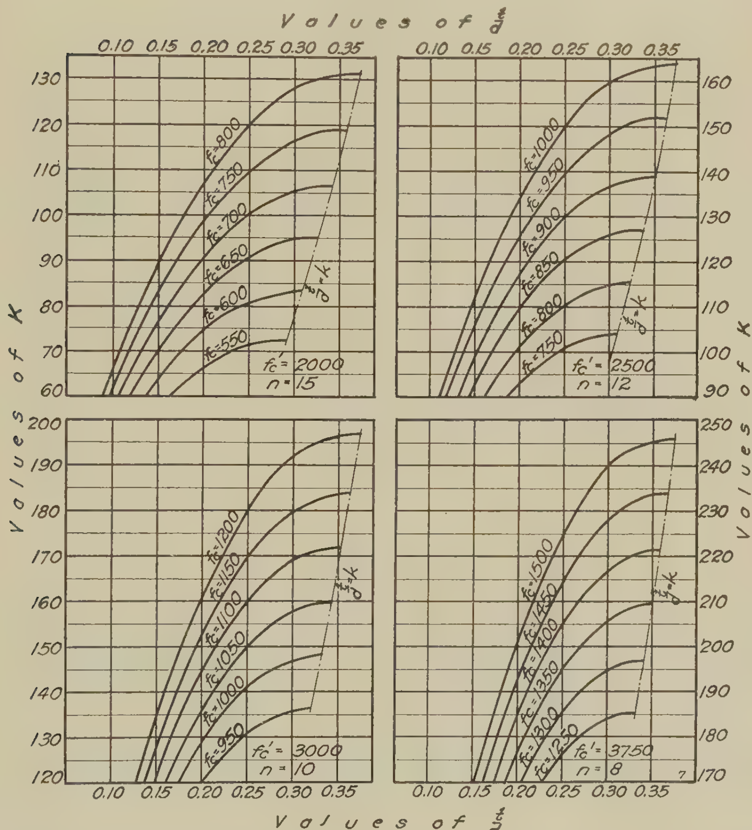
DIAGRAM 6.—RECTANGULAR BEAMS. f_c VARIES



INSTRUCTIONS FOR USE.—Use this diagram to determine the actual concrete stress (which is less than the maximum allowable value with balanced reinforcement), when the steel area has been determined by formula (103b) as described on page 542. Determine the value of K by formula (104), page 546, and read off the concrete stress at the intersection of a horizontal line through the value of K with the sloping scale of stresses.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

DIAGRAM 7.—TEE BEAMS. f_c VARIES



INSTRUCTIONS FOR USE.—Use this diagram to determine the actual stress (which is less than the maximum allowable value with balanced reinforcement), when the steel area has been determined by formula (103b) as described on page 542. Determine the value of K by formula (104), page 546, and the value of t/d from the known slab thickness and assumed beam depth. Read off the concrete stress between the curved scales at the intersection of a vertical line through the value of t/d with a horizontal line through the value of K.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 8.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.40f'_c$

p'	2000-lb. Concrete $n = 15$ $f_c = 800$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0086	152	0.0085	150	0.0084	149	0.0084	147	0.0083	146
0.004	0.0096	173	0.0095	169	0.0094	166	0.0093	163	0.0092	161
0.006	0.0107	193	0.0105	189	0.0103	184	0.0101	180	0.0100	175
0.008	0.0117	214	0.0115	208	0.0113	202	0.0110	196	0.0108	180
0.010	0.0128	235	0.0125	227	0.0122	219	0.0119	212	0.0116	205
0.012	0.0139	256	0.0135	246	0.0131	237	0.0128	228	0.0124	220
0.014	0.0149	276	0.0145	265	0.0141	255	0.0137	244	0.0132	234
0.016	0.0160	297	0.0155	285	0.0150	272	0.0145	261	0.0141	249
0.018	0.0170	318	0.0165	304	0.0160	290	0.0154	277	0.0149	264
0.020	0.0181	339	0.0175	323	0.0169	308	0.0163	293	0.0157	279
0.022	0.0192	360	0.0185	342	0.0178	325	0.0172	309	0.0165	294
0.024	0.0202	380	0.0195	361	0.0188	343	0.0181	325	0.0174	308
0.026	0.0213	401	0.0205	381	0.0197	361	0.0189	342	0.0182	323
0.028	0.0223	422	0.0215	400	0.0207	379	0.0198	358	0.0190	338
0.030	0.0234	442	0.0225	419	0.0216	396	0.0207	374	0.0198	353

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0083	144	0.0082	143	0.0081	142	0.0081	141	0.0080	139
0.004	0.0090	158	0.0089	155	0.0088	153	0.0087	150	0.0085	148
0.006	0.0098	171	0.0096	167	0.0094	163	0.0092	160	0.0091	156
0.008	0.0105	185	0.0103	179	0.0101	174	0.0098	169	0.0096	164
0.010	0.0113	198	0.0110	191	0.0107	185	0.0104	179	0.0101	173
0.012	0.0121	211	0.0117	203	0.0113	196	0.0110	188	0.0106	181
0.014	0.0128	225	0.0124	216	0.0120	207	0.0116	198	0.0112	190
0.016	0.0136	238	0.0131	228	0.0126	217	0.0122	207	0.0117	198
0.018	0.0143	252	0.0138	240	0.0133	228	0.0127	217	0.0122	206
0.020	0.0151	265	0.0145	252	0.0139	239	0.0133	226	0.0127	215
0.022	0.0159	278	0.0152	264	0.0145	250	0.0139	236	0.0132	223
0.024	0.0166	292	0.0159	276	0.0152	261	0.0145	246	0.0138	231
0.026	0.0174	305	0.0166	288	0.0158	271	0.0151	255	0.0143	240
0.028	0.0181	319	0.0173	300	0.0165	282	0.0157	265	0.0148	248
0.030	0.0189	332	0.0180	312	0.0171	293	0.0162	274	0.0153	256

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 2000-lb. concrete and having compressive reinforcement, except for the special case covered by Table 9. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 9.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.45f'_c$

p'	2000-lb. Concrete $n = 15$ $fc = 900$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0103	180	0.0102	179	0.0102	177	0.0101	176	0.0100	174
0.004..........	0.0115	204	0.0114	200	0.0112	197	0.0111	194	0.0110	191
0.006..........	0.0127	227	0.0125	222	0.0123	218	0.0121	213	0.0119	208
0.008..........	0.0139	251	0.0136	244	0.0134	238	0.0131	231	0.0129	225
0.010..........	0.0151	274	0.0148	266	0.0145	258	0.0141	250	0.0138	242
0.012..........	0.0163	298	0.0159	288	0.0155	278	0.0151	268	0.0148	259
0.014..........	0.0175	321	0.0170	310	0.0166	298	0.0161	287	0.0157	276
0.016..........	0.0187	345	0.0181	331	0.0177	319	0.0172	306	0.0167	293
0.018..........	0.0199	368	0.0193	353	0.0188	339	0.0182	324	0.0176	310
0.020..........	0.0211	392	0.0204	375	0.0198	359	0.0192	343	0.0186	328
0.022..........	0.0223	415	0.0215	397	0.0209	379	0.0202	361	0.0195	345
0.024..........	0.0235	438	0.0226	419	0.0219	399	0.0212	380	0.0205	362
0.026..........	0.0247	462	0.0238	440	0.0230	419	0.0222	398	0.0214	379
0.028..........	0.0259	485	0.0249	462	0.0241	439	0.0232	417	0.0224	396
0.030..........	0.0271	509	0.0261	484	0.0252	459	0.0242	436	0.0233	413

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0100	173	0.0099	171	0.0099	170	0.0098	168	0.0097	167
0.004..........	0.0109	188	0.0107	185	0.0106	182	0.0105	180	0.0104	177
0.006..........	0.0117	204	0.0116	199	0.0114	195	0.0112	191	0.0110	187
0.008..........	0.0126	219	0.0124	214	0.0121	208	0.0119	203	0.0116	198
0.010..........	0.0135	235	0.0132	228	0.0129	221	0.0126	214	0.0123	208
0.012..........	0.0144	250	0.0140	242	0.0136	234	0.0133	226	0.0129	218
0.014..........	0.0153	266	0.0148	256	0.0144	246	0.0140	237	0.0135	228
0.016..........	0.0161	282	0.0157	270	0.0152	259	0.0147	249	0.0142	238
0.018..........	0.0170	297	0.0165	284	0.0159	272	0.0154	260	0.0148	248
0.020..........	0.0179	313	0.0173	298	0.0167	285	0.0161	271	0.0154	259
0.022..........	0.0188	328	0.0181	312	0.0175	297	0.0168	283	0.0161	269
0.024..........	0.0197	344	0.0189	327	0.0182	310	0.0175	294	0.0167	279
0.026..........	0.0206	359	0.0197	341	0.0190	323	0.0182	306	0.0173	289
0.028..........	0.0215	375	0.0206	355	0.0197	336	0.0189	317	0.0180	299
0.030..........	0.0224	391	0.0214	369	0.0205	349	0.0196	329	0.0186	309

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 2000-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 10.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.40f'_c$

p'	2500-lb. Concrete $n = 12$ $f_c = 1000$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0104	185	0.0104	183	0.0103	181	0.0103	180	0.0102	179
0.004	0.0115	205	0.0114	202	0.0112	199	0.0111	196	0.0110	193
0.006	0.0125	225	0.0123	221	0.0122	216	0.0120	212	0.0118	208
0.008	0.0136	246	0.0133	239	0.0131	234	0.0129	228	0.0126	222
0.010	0.0146	266	0.0143	258	0.0140	251	0.0137	244	0.0134	237
0.012	0.0156	286	0.0153	277	0.0149	268	0.0146	260	0.0142	251
0.014	0.0167	307	0.0163	296	0.0159	286	0.0155	275	0.0150	266
0.016	0.0177	327	0.0172	314	0.0168	303	0.0163	291	0.0159	280
0.018	0.0188	348	0.0182	334	0.0177	321	0.0172	307	0.0167	295
0.020	0.0198	368	0.0192	352	0.0186	338	0.0181	323	0.0175	309
0.022	0.0208	388	0.0202	371	0.0196	355	0.0189	339	0.0183	324
0.024	0.0219	408	0.0212	390	0.0205	373	0.0198	355	0.0191	338
0.026	0.0229	429	0.0221	409	0.0214	390	0.0207	371	0.0199	353
0.028	0.0239	449	0.0231	428	0.0223	408	0.0215	387	0.0207	367
0.030	0.0250	470	0.0241	447	0.0233	425	0.0224	403	0.0215	382

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0101	177	0.0101	176	0.0100	175	0.0100	173	0.0099	172
0.004	0.0109	190	0.0108	188	0.0107	185	0.0105	183	0.0104	180
0.006	0.0116	203	0.0115	200	0.0113	196	0.0111	192	0.0109	189
0.008	0.0124	217	0.0122	211	0.0119	206	0.0117	201	0.0115	197
0.010	0.0131	230	0.0128	223	0.0126	217	0.0123	211	0.0120	205
0.012	0.0139	243	0.0135	235	0.0132	227	0.0128	220	0.0125	213
0.014	0.0146	256	0.0142	247	0.0138	238	0.0134	230	0.0130	221
0.016	0.0154	269	0.0149	259	0.0144	249	0.0140	239	0.0135	230
0.018	0.0161	283	0.0156	271	0.0151	259	0.0145	248	0.0140	238
0.020	0.0169	295	0.0163	283	0.0157	270	0.0151	258	0.0145	246
0.022	0.0176	309	0.0170	295	0.0163	281	0.0157	267	0.0150	254
0.024	0.0184	322	0.0177	306	0.0170	291	0.0163	276	0.0156	263
0.026	0.0191	335	0.0184	318	0.0176	302	0.0168	285	0.0161	271
0.028	0.0199	349	0.0191	330	0.0182	312	0.0174	295	0.0166	279
0.030	0.0206	362	0.0197	342	0.0189	323	0.0180	304	0.0171	287

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 2500-lb. concrete and having compressive reinforcement, except for the special case covered by Table 11. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $fs = 20{,}000$

TABLE 11.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $fc = 0.45f'c$

p'	2500-lb. Concrete $n = 12$ $fc = 1125$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0125	219	0.0124	217	0.0124	216	0.0123	214	0.0122	213
0.004..........	0.0137	242	0.0135	238	0.0134	236	0.0133	232	0.0132	229
0.006..........	0.0148	265	0.0146	260	0.0145	255	0.0143	251	0.0141	246
0.008..........	0.0160	288	0.0158	282	0.0155	275	0.0153	269	0.0150	263
0.010..........	0 0172	311	0.0169	303	0.0166	295	0.0163	287	0.0160	280
0.012..........	0.0183	334	0.0180	324	0.0176	315	0.0172	305	0.0169	296
0.014..........	0.0195	357	0.0191	346	0.0187	335	0.0182	324	0.0178	313
0 016..........	0.0207	380	0.0202	367	0.0197	354	0.0192	342	0.0187	330
0.018..........	0.0219	403	0.0213	389	0.0208	374	0.0202	360	0.0197	347
0.020..........	0.0230	426	0.0224	410	0 0218	394	0.0212	378	0.0206	363
0.022..........	0.0242	450	0.0236	431	0.0229	414	0.0222	397	0.0215	380
0 024..........	0.0254	473	0.0247	452	0.0239	434	0.0232	415	0.0225	397
0 026..........	0.0266	496	0.0258	474	0.0250	453	0.0242	433	0.0234	414
0 028..........	0 0277	519	0.0269	496	0.0260	473	0.0252	451	0.0243	430
0.030..........	0 0289	542	0.0280	517	0.0271	493	0.0262	470	0.0253	447

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0122	211	0.0121	210	0.0120	209	0.0120	207	0.0119	206
0.004..........	0 0130	227	0.0129	224	0.0128	221	0.0127	218	0.0125	216
0 006	0 0139	242	0.0137	238	0.0135	234	0.0133	230	0.0132	226
0.008..........	0.0148	257	0.0145	252	0.0143	246	0.0140	241	0.0138	236
0.010..........	0.0156	272	0.0153	265	0.0150	259	0.0147	252	0.0144	246
0.012..........	0.0165	288	0.0161	279	0.0157	271	0.0154	263	0.0150	256
0.014..........	0.0174	303	0.0169	293	0.0165	284	0.0161	275	0.0157	266
0.016..........	0.0183	318	0.0178	307	0.0172	296	0.0167	286	0.0163	276
0.018..........	0 0192	334	0.0186	321	0.0180	309	0.0174	297	0.0169	286
0.020..........	0.0200	349	0.0194	335	0.0187	321	0.0181	308	0.0175	296
0.022..........	0.0208	364	0.0202	349	0.0194	334	0.0188	320	0.0182	306
0.024..........	0.0216	379	0.0210	363	0.0202	346	0.0195	331	0.0188	316
0.026..........	0.0225	395	0.0218	376	0.0209	359	0.0202	342	0.0194	326
0.028..........	0.0234	410	0.0226	390	0.0217	371	0.0209	353	0.0200	336
0.030..........	0.0243	425	0.0234	404	0.0225	384	0.0216	364	0.0206	346

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 2500-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 12.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT. $f_c = 0.40f'_c$

3000-lb. Concrete $n = 10$ $f_c = 1200$

p'	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0122	217	0.0122	216	0.0121	214	0.0120	213	0.0120	211
0.004	0.0132	237	0.0131	234	0.0130	231	0.0129	228	0.0128	226
0.006	0.0143	257	0.0141	252	0.0139	248	0.0137	244	0.0136	240
0.008	0.0153	277	0.0151	271	0.0148	265	0.0146	260	0.0144	254
0.010	0.0163	297	0.0160	290	0.0158	282	0.0154	275	0.0152	268
0.012	0.0173	317	0.0170	308	0.0167	299	0.0163	291	0.0160	283
0.014	0.0183	337	0.0179	327	0.0176	316	0.0171	306	0.0167	297
0.016	0.0194	357	0.0189	345	0.0185	333	0.0180	322	0.0175	311
0.018	0.0204	377	0.0198	364	0.0194	350	0.0188	338	0.0183	325
0.020	0.0214	397	0.0208	382	0.0203	367	0.0197	353	0.0191	340
0.022	0.0224	417	0.0218	401	0.0212	385	0.0205	369	0.0199	354
0.024	0.0234	437	0.0227	419	0.0221	402	0.0214	385	0.0207	368
0.026	0.0245	457	0.0237	438	0.0230	419	0.0222	400	0.0215	382
0.028	0.0255	477	0.0247	456	0.0239	436	0.0231	416	0.0223	397
0.030	0.0265	497	0.0257	475	0.0248	453	0.0239	432	0.0231	411

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0119	210	0.0119	209	0.0118	207	0.0118	206	0.0117	205
0.004	0.0127	223	0.0125	220	0.0124	218	0.0123	215	0.0122	213
0.006	0.0134	236	0.0132	232	0.0131	228	0.0129	225	0.0127	221
0.008	0.0141	249	0.0139	243	0.0137	239	0.0134	234	0.0132	229
0.010	0.0149	262	0.0146	255	0.0143	249	0.0140	243	0.0137	237
0.012	0.0156	275	0.0153	267	0.0149	259	0.0146	252	0.0142	245
0.014	0.0163	287	0.0159	278	0.0155	270	0.0151	261	0.0147	253
0.016	0.0171	300	0.0166	290	0.0162	280	0.0157	271	0.0152	262
0.018	0.0178	313	0.0173	302	0.0168	291	0.0162	280	0.0157	270
0.020	0.0185	326	0.0180	313	0.0174	301	0.0168	289	0.0162	278
0.022	0.0193	339	0.0186	325	0.0180	311	0.0174	298	0.0167	286
0.024	0.0200	352	0.0193	337	0.0186	322	0.0179	307	0.0172	294
0.026	0.0207	365	0.0200	348	0.0191	332	0.0185	317	0.0178	302
0.028	0.0215	378	0.0207	360	0.0199	343	0.0191	326	0.0183	310
0.030	0.0222	391	0.0214	372	0.0205	353	0.0196	335	0.0188	318

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 3000-lb. concrete and having compressive reinforcement except for the special case covered by Table 13. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 13.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.45 f'_c$

p'	3000-lb. Concrete $n = 10$ $f_c = 1350$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0148	258	0.0147	256	0.0146	254	0.0146	253	0.0145	251
0.004..........	0.0159	280	0.0158	277	0.0157	274	0.0155	271	0.0154	268
0.006..........	0.0171	303	0.0169	298	0.0167	293	0.0165	289	0.0163	284
0.008..........	0.0182	325	0.0180	319	0.0177	313	0.0175	307	0.0173	301
0.010..........	0.0194	348	0.0191	340	0.0188	332	0.0185	325	0.0182	317
0.012..........	0.0205	371	0.0201	361	0.0198	352	0.0194	342	0.0191	334
0.014..........	0.0217	393	0.0212	382	0.0208	371	0.0204	360	0.0200	350
0.016..........	0.0228	416	0.0223	403	0.0218	391	0.0214	378	0.0209	367
0.018..........	0.0240	439	0.0234	424	0.0229	410	0.0223	396	0.0218	383
0.020..........	0.0251	461	0.0245	445	0.0239	429	0.0233	414	0.0227	399
0.022..........	0.0263	484	0.0256	466	0.0249	449	0.0243	432	0.0236	416
0.024..........	0.0274	506	0.0267	487	0.0260	468	0.0252	450	0.0246	432
0.026..........	0.0286	529	0.0278	508	0.0270	488	0.0262	468	0.0255	449
0.028..........	0.0297	552	0.0289	529	0.0280	507	0.0272	485	0.0264	465
0.030..........	0.0309	574	0.0300	550	0.0291	527	0.0282	504	0.0273	482

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0145	250	0.0144	249	0.0143	247	0.0143	246	0.0142	245
0.004..........	0.0153	265	0.0152	262	0.0151	260	0.0149	257	0.0148	255
0.006..........	0.0162	280	0.0160	276	0.0158	272	0.0156	268	0.0154	264
0.008..........	0.0170	295	0.0168	290	0.0165	284	0.0163	279	0.0160	274
0.010..........	0.0179	310	0.0176	303	0.0173	297	0.0170	290	0.0167	284
0.012..........	0.0187	325	0.0183	317	0.0180	309	0.0176	301	0.0173	294
0.014..........	0.0196	340	0.0191	330	0.0187	321	0.0183	312	0.0179	304
0.016..........	0.0204	355	0.0199	344	0.0194	333	0.0190	323	0.0185	313
0.018..........	0.0213	370	0.0207	358	0.0202	346	0.0196	334	0.0191	323
0.020..........	0.0221	385	0.0215	371	0.0209	358	0.0203	345	0.0197	333
0.022..........	0.0230	400	0.0223	385	0.0216	370	0.0210	356	0.0203	343
0.024..........	0.0238	415	0.0231	399	0.0224	383	0.0216	367	0.0209	353
0.026..........	0.0247	430	0.0239	412	0.0231	395	0.0223	378	0.0216	362
0.028..........	0.0255	445	0.0247	426	0.0238	407	0.0230	389	0.0222	372
0.030..........	0.0264	460	0.0255	440	0.0246	420	0.0237	400	0.0228	382

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 3000-lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 14.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$f_c = 0.40 f'_c$

3750-lb. Concrete $n = 8$ $f_c = 1500$

p'	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0151	265	0.0150	264	0.0150	263	0.0149	261	0.0149	260
0.004..........	0.0161	285	0.0160	282	0.0159	279	0.0158	276	0.0156	274
0.006..........	0.0171	304	0.0169	300	0.0167	296	0.0166	292	0.0164	288
0.008..........	0.0181	324	0.0179	318	0.0176	312	0.0174	307	0.0172	301
0.010..........	0.0191	343	0.0188	336	0.0185	329	0.0182	322	0.0179	315
0.012..........	0.0201	363	0.0197	354	0.0194	346	0.0191	337	0.0187	329
0.014..........	0.0211	382	0.0207	372	0.0203	362	0.0199	352	0.0195	343
0.016..........	0.0221	402	0.0216	390	0.0212	379	0.0207	368	0.0203	357
0.018..........	0.0230	421	0.0225	408	0.0220	395	0.0215	383	0.0210	371
0.020..........	0.0240	441	0.0235	426	0.0229	412	0.0224	398	0.0218	385
0.022..........	0.0250	460	0.0244	444	0.0238	429	0.0232	413	0.0226	398
0.024..........	0.0260	480	0.0253	462	0.0247	445	0.0240	428	0.0233	412
0.026..........	0.0270	499	0.0263	480	0.0256	462	0.0248	444	0.0241	426
0.028..........	0.0280	518	0.0272	498	0.0265	478	0.0257	459	0.0249	440
0.030..........	0.0290	538	0.0282	516	0.0273	495	0.0265	474	0.0256	454

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0148	259	0.0148	257	0.0147	256	0.0146	255	0.0146	254
0.004..........	0.0155	271	0.0154	269	0.0153	266	0.0152	264	0.0151	262
0.006..........	0.0162	284	0.0161	280	0.0159	276	0.0157	273	0.0156	270
0.008..........	0.0170	296	0.0167	291	0.0165	286	0.0163	282	0.0161	277
0.010..........	0.0177	309	0.0174	303	0.0171	297	0.0168	291	0.0166	285
0.012..........	0.0184	321	0.0180	314	0.0177	307	0.0174	300	0.0170	293
0.014..........	0.0191	334	0.0187	325	0.0183	317	0.0179	309	0.0175	301
0.016..........	0.0198	346	0.0194	337	0.0189	327	0.0185	318	0.0180	309
0.018..........	0.0205	359	0.0200	348	0.0195	337	0.0190	327	0.0185	317
0.020..........	0.0212	372	0.0207	359	0.0201	347	0.0196	336	0.0190	324
0.022..........	0.0220	384	0.0213	371	0.0207	357	0.0201	345	0.0195	332
0.024..........	0.0227	397	0.0220	382	0.0213	367	0.0207	354	0.0200	340
0.026..........	0.0234	409	0.0227	393	0.0219	378	0.0212	362	0.0205	348
0.028..........	0.0241	422	0.0233	404	0.0225	388	0.0217	371	0.0210	356
0.030..........	0.0248	434	0.0240	416	0.0231	398	0.0223	380	0.0214	364

INSTRUCTIONS FOR USE.—This table is to be used in beams and slabs made of 3750-lb. concrete and having compressive reinforcement, except for the special case covered by Table 15. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 15.—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT.

$fc = 0.45 f'c$

p'	3750-lb. Concrete $n = 8$ $fc = 1688$									
	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0181	316	0.0181	314	0.0180	313	0.0180	311	0.0179	310
0.004	0.0192	338	0.0191	335	0.0190	332	0.0189	329	0.0188	326
0.006	0.0204	360	0.0202	355	0.0200	351	0.0198	346	0.0197	342
0.008	0.0215	382	0.0212	376	0.0210	370	0.0208	364	0.0206	358
0.010	0.0226	404	0.0223	396	0.0220	388	0.0217	381	0.0214	374
0.012	0.0237	426	0.0234	416	0.0230	407	0.0226	399	0.0223	390
0.014	0.0249	448	0.0244	437	0.0240	426	0.0236	416	0.0232	406
0.016	0.0260	470	0.0255	457	0.0250	445	0.0245	433	0.0241	422
0.018	0.0271	492	0.0265	478	0.0261	465	0.0255	451	0.0250	438
0.020	0.0282	514	0.0276	498	0.0271	484	0.0265	468	0.0259	454
0.022	0.0293	536	0.0287	518	0.0281	503	0.0274	486	0.0268	470
0.024	0.0304	558	0.0398	539	0.0291	522	0.0284	503	0.0277	486
0.026	0.0316	580	0.0308	559	0.0301	541	0.0293	521	0.0286	502
0.028	0.0327	602	0.0319	580	0.0311	560	0.0303	538	0.0294	518
0.030	0.0338	624	0.0330	600	0.0321	578	0.0312	556	0.0303	534

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002	0.0178	309	0.0178	307	0.0177	306	0.0177	305	0.0176	304
0.004	0.0187	323	0.0185	321	0.0184	318	0.0183	315	0.0182	313
0.006	0.0195	338	0.0193	334	0.0191	330	0.0189	326	0.0188	323
0.008	0.0203	352	0.0201	347	0.0199	342	0.0196	337	0.0194	332
0.010	0.0211	367	0.0208	360	0.0206	354	0.0202	348	0.0200	342
0.012	0.0220	382	0.0216	374	0.0213	366	0.0209	358	0.0206	351
0.014	0.0228	396	0.0224	387	0.0220	378	0.0215	369	0.0212	361
0.016	0.0236	410	0.0232	400	0.0227	390	0.0222	380	0.0218	370
0.018	0.0245	425	0.0339	413	0.0234	401	0.0228	390	0.0224	380
0.020	0.0253	440	0.0247	427	0.0241	414	0.0235	401	0.0230	389
0.022	0.0261	455	0.0255	440	0.0248	426	0.0242	412	0.0235	399
0.024	0.0270	469	0.0262	453	0.0255	438	0.0248	423	0.0241	408
0.026	0.0278	484	0.0270	466	0.0263	449	0.0255	433	0.0247	418
0.028	0.0286	498	0.0278	480	0.0270	461	0.0262	444	0.0253	427
0.030	0.0294	513	0.0286	493	0.0277	473	0.0268	455	0.0259	437

INSTRUCTIONS FOR USE.—This table is to be used only at sections adjacent to the supports of continuous or fixed beams or of rigid frames, made of 3750 lb. concrete and having compressive reinforcement. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR f_s = 20,000

TABLE 16.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

2000-lb. Concrete $n = 15$ $fc = 800$

$\frac{t}{d}$	p	K
0 04	0.0015	30
0.06	0.0022	43
0.08	0.0029	55
0.10	0.0035	66
0 12	0.0040	76
0.14	0.0046	85
0.16	0.0050	93
0.18	0.0055	101
0.20	0.0059	107
0.22	0.0062	113
0.24	0.0065	117
0.26	0.0068	121
0.28	0.0070	125
0.30	0.0072	128
0.32	0.0073	129
0.34	0.0074	130
0.36	0.0075	131
k	0.0075	131

p'	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0 0009	18	0 0009	16	0.0008	15
0 004	0.0020	38	0 0019	35	0 0018	32	0.0016	30
0.006	0 0030	58	0 0028	53	0 0026	49	0 0025	44
0.008	0.0040	77	0.0038	71	0 0035	65	0 0033	59
0.010	0.0050	96	0.0047	88	0 0044	81	0.0041	74
0.012	0.0060	115	0.0056	106	0 0053	97	0.0049	89
0.014	0.0070	134	0.0066	124	0.0062	113	0 0057	103
0.016	0.0080	154	0.0075	141	0.0070	130	0.0066	118
0.018	0.0090	173	0.0085	159	0.0079	146	0 0074	133
0.020	0.0100	192	0.0094	177	0.0088	162	0 0082	148
0.022	0.0110	211	0.0103	194	0.0097	178	0 0090	163
0.024	0.0120	230	0.0113	212	0.0106	194	0 0099	177
0.026	0.0130	250	0.0122	230	0.0114	211	0.0107	192
0.028	0.0140	269	0.0132	248	0.0123	227	0 0115	207
0.030	0.0150	288	0.0141	265	0.0132	243	0 0123	222

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0008	13	0.0007	12	0.0006	11	0.0006	10	0.0005	
0.004..........	0.0015	27	0.0014	24	0.0013	21	0.0012	19	0.0010	17
0.006..........	0.0023	40	0.0021	36	0.0019	32	0.0017	29	0.0016	25
0.008..........	0.0030	54	0.0028	48	0.0026	43	0.0023	38	0.0021	33
0.010..........	0.0038	67	0.0035	60	0.0032	54	0.0029	48	0.0026	42
0.012..........	0.0046	80	0.0042	72	0.0038	65	0.0035	57	0.0031	50
0.014..........	0.0053	94	0.0049	85	0.0045	75	0.0041	67	0.0037	59
0.016..........	0.0061	107	0.0056	97	0.0051	86	0.0047	76	0.0042	67
0.018..........	0.0068	121	0.0063	109	0.0058	97	0.0052	86	0.0047	75
0.020..........	0.0076	134	0.0070	121	0.0064	108	0.0058	95	0.0052	84
0.022..........	0.0084	147	0.0077	133	0.0070	118	0.0064	105	0.0057	92
0.024..........	0.0091	161	0.0084	145	0.0077	129	0.0070	114	0.0063	100
0.026..........	0.0099	174	0.0091	157	0.0083	140	0.0075	124	0.0068	109
0.028..........	0.0106	188	0.0098	169	0.0090	151	0.0081	133	0.0073	117
0.030..........	0.0114	201	0.0105	181	0.0096	162	0.0087	143	0.0078	125

GENERAL NOTE FOR TABLES 16 TO 19

INSTRUCTIONS FOR USE.—Ordinarily T-beams require no compressive reinforcement but Tables 16 to 19 will be found useful where architectural considerations require certain T-joists or beams to carry extraordinary loads without increase in depth, or require the removal of portions of the flange. (See also general note under Table 5.)

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 17.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

$\frac{t}{d}$	p	K
0.04	0.0019	37
0.06	0.0028	54
0.08	0.0036	69
0.10	0.0043	83
0.12	0.0050	95
0.14	0.0057	107
0.16	0.0063	117
0.18	0.0068	126
0.20	0.0073	134
0.22	0.0078	141
0.24	0.0082	147
0.26	0.0085	152
0.28	0.0088	156
0.30	0.0090	160
0.32	0.0092	162
0.34	0.0093	163
0.36	0.0094	164
k	0.0094	164

p'	2500-lb. Concrete $n = 12$ $fc = 1000$							
	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0.0009	17	0.0009	16	0.0008	15
0.004	0.0020	38	0.0018	35	0.0017	32	0.0016	29
0.006	0.0029	57	0.0028	52	0.0026	48	0.0024	44
0.008	0.0039	75	0.0037	70	0.0035	64	0.0032	58
0.010	0.0049	94	0.0046	87	0.0043	80	0.0040	73
0.012	0.0059	113	0.0055	104	0.0052	96	0.0048	87
0.014	0.0069	132	0.0065	122	0.0061	112	0.0056	102
0.016	0.0079	151	0.0074	139	0.0069	127	0.0065	116
0.018	0.0088	170	0.0083	157	0.0078	143	0.0073	131
0.020	0.0098	189	0.0092	174	0.0087	159	0.0081	145
0.022	0.0108	207	0.0102	191	0.0095	175	0.0089	160
0.024	0.0118	226	0.0111	209	0.0104	191	0.0097	174
0.026	0.0128	245	0.0120	226	0.0113	207	0.0105	189
0.028	0.0138	264	0.0129	244	0.0121	223	0.0113	203
0.030	0.0147	283	0.0139	261	0.0130	239	0.0121	218

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0007	13	0.0007	12	0.0006	11	0.0006	9	0.0005	8
0.004..........	0.0015	26	0.0014	24	0.0013	21	0.0011	19	0.0010	16
0.006..........	0.0022	40	0.0021	36	0.0019	32	0 0017	28	0.0015	25
0.008..........	0.0030	53	0.0028	47	0.0025	42	0.0023	37	0.0021	33
0.010..........	0.0037	66	0.0034	59	0.0032	53	0.0029	47	0.0026	41
0.012..........	0.0045	79	0.0041	71	0.0038	64	0.0034	56	0.0031	49
0.014..........	0.0052	92	0.0048	83	0.0044	74	0.0040	66	0.0036	57
0.016..........	0.0060	105	0.0055	95	0.0050	85	0.0046	75	0.0041	66
0.018..........	0.0067	119	0.0062	107	0.0057	95	0.0051	84	0.0046	74
0.020..........	0.0075	132	0.0069	119	0.0063	106	0.0057	94	0 0051	82
0.022..........	0.0082	145	0.0076	130	0.0069	117	0.0063	103	0.0056	90
0.024..........	0.0090	158	0.0083	142	0.0076	127	0.0069	112	0.0062	99
0.026..........	0.0097	171	0.0090	154	0.0082	138	0.0074	122	0.0067	107
0.028..........	0.0105	185	0.0097	166	0.0088	148	0.0080	131	0.0072	115
0.030..........	0.0112	198	0.0103	178	0.0095	159	0.0086	140	0.0077	123

See instructions for use under Table 16.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 18.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

$\frac{t}{d}$	p	K
0.04	0.0023	45
0.06	0.0033	64
0.08	0.0043	82
0.10	0.0052	99
0.12	0.0061	114
0.14	0.0068	128
0.16	0.0076	140
0.18	0.0082	151
0.20	0.0088	161
0.22	0.0093	169
0.24	0.0098	176
0.26	0.0102	182
0.28	0.0105	187
0.30	0.0108	192
0.32	0.0110	194
0.34	0.0112	196
0.36	0.0112	197
k	0.0113	197

p'	3000-lb. Concrete $n = 10$ $fc = 1200$							
	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0010	19	0.0009	17	0.0008	16	0.0008	14
0.004	0.0019	37	0.0018	34	0.0017	31	0.0016	29
0.006	0.0029	56	0.0027	51	0.0025	47	0.0024	43
0.008	0.0039	74	0.0036	68	0.0034	63	0.0032	57
0.010	0.0048	93	0.0045	85	0.0042	78	0.0040	71
0.012	0.0058	111	0.0054	102	0.0051	94	0.0048	86
0.014	0.0068	130	0.0064	119	0.0059	109	0.0056	100
0.016	0.0077	148	0.0073	136	0.0068	125	0.0063	114
0.018	0.0087	167	0.0082	154	0.0076	141	0.0071	128
0.020	0.0096	185	0.0091	171	0.0085	156	0.0079	143
0.022	0.0106	204	0.0100	188	0.0093	172	0.0087	157
0.024	0.0116	222	0.0109	205	0.0102	188	0.0095	171
0.026	0.0125	241	0.0118	222	0.0110	204	0.0103	185
0.028	0.0135	260	0.0127	239	0.0119	219	0.0111	200
0.030	0.0145	278	0.0136	256	0.0127	235	0.0119	214

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0007	13	0.0007	12	0.0006	10	0.0006	9	0.0005	8
0.004..........	0.0015	26	0.0014	23	0.0012	21	0.0011	18	0.0010	16
0.006..........	0.0022	39	0.0020	35	0.0019	31	0.0017	28	0.0015	24
0.008..........	0.0029	52	0.0027	47	0.0025	42	0.0022	37	0.0020	32
0.010..........	0.0037	65	0.0034	58	0.0031	52	0.0028	46	0.0025	40
0.012..........	0.0044	78	0.0041	70	0.0037	62	0.0034	55	0.0030	48
0.014..........	0.0051	90	0.0047	81	0.0043	73	0.0039	65	0.0035	56
0.016..........	0.0059	103	0.0054	93	0.0050	83	0.0045	74	0.0040	64
0.018..........	0.0066	116	0.0061	105	0.0056	94	0.0051	83	0.0045	73
0.020..........	0.0073	129	0.0068	116	0.0062	104	0.0056	92	0.0050	81
0.022..........	0.0081	142	0.0074	128	0.0068	114	0.0062	101	0.0055	89
0.024..........	0.0088	155	0.0081	140	0.0074	125	0.0067	111	0.0060	97
0.026..........	0.0095	168	0.0088	151	0.0080	135	0.0073	120	0.0066	105
0.028..........	0.0103	181	0.0095	163	0.0087	146	0.0079	129	0.0071	113
0.030..........	0.0110	194	0.0102	175	0.0093	156	0.0084	138	0.0076	121

See instructions for use under Table 16.

DESIGN VALUES OF p AND K FOR $f_s = 20{,}000$

TABLE 19.—T-BEAMS WITH COMPRESSIVE REINFORCEMENT

3750-lb. Concrete $n = 8$ $f_c = 1500$

$\frac{t}{d}$	p	K
0.04	0.0028	56
0.06	0.0041	80
0.08	0.0054	103
0.10	0.0065	124
0.12	0.0076	143
0.14	0.0085	160
0.16	0.0094	175
0.18	0.0103	189
0.20	0.0110	201
0.22	0.0117	211
0.24	0.0122	220
0.26	0.0127	228
0.28	0.0132	234
0.30	0.0135	240
0.32	0.0138	242
0.34	0.0139	245
0.36	0.0140	246
k	0.0140	246

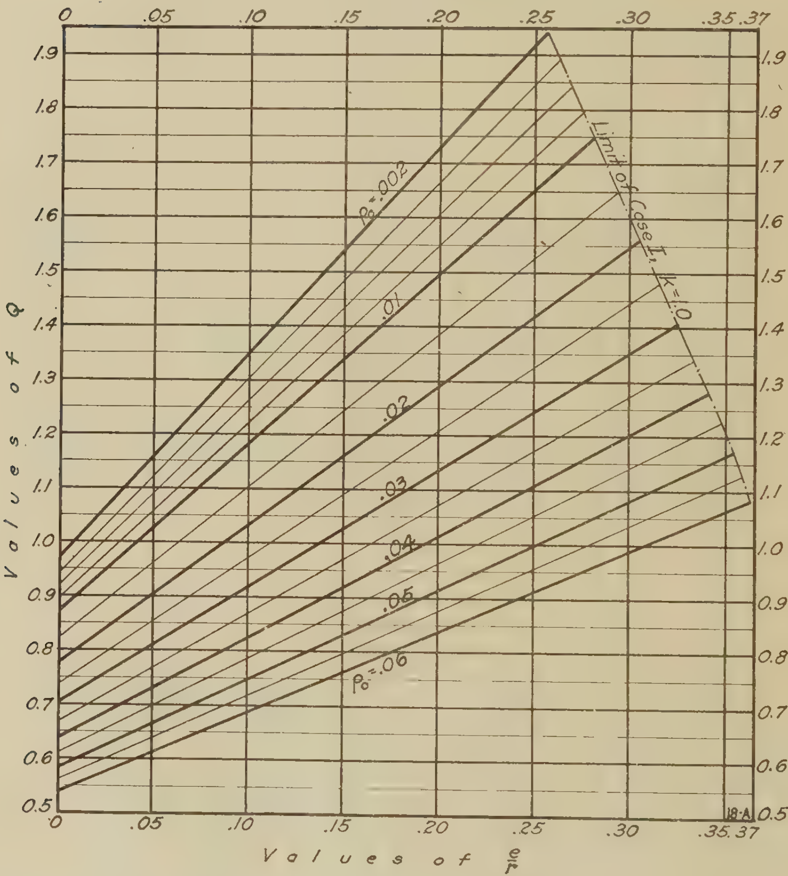
p'	$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	p	K	p	K	p	K	p	K
0.002	0.0009	18	0.0009	17	0.0008	15	0.0008	14
0.004	0.0019	36	0.0018	33	0.0017	30	0.0015	28
0.006	0.0028	54	0.0026	50	0.0025	46	0.0023	42
0.008	0.0038	72	0.0035	66	0.0033	61	0.0031	55
0.010	0.0047	90	0.0044	83	0.0041	76	0.0038	69
0.012	0.0056	108	0.0053	100	0.0050	91	0.0046	83
0.014	0.0066	126	0.0062	116	0.0058	106	0.0054	97
0.016	0.0075	144	0.0071	133	0.0066	122	0.0062	111
0.018	0.0084	162	0.0079	149	0.0074	137	0.0069	125
0.020	0.0094	180	0.0088	166	0.0083	152	0.0077	139
0.022	0.0103	198	0.0097	183	0.0091	167	0.0085	152
0.024	0.0112	216	0.0106	199	0.0099	182	0.0092	166
0.026	0.0122	234	0.0115	216	0.0107	198	0.0100	180
0.028	0.0131	252	0.0124	232	0.0116	213	0.0108	194
0.030	0.0141	270	0.0132	249	0.0124	228	0.0116	208

p'	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	p	K	p	K	p	K	p	K	p	K
0.002..........	0.0007	13	0.0007	11	0.0006	10	0.0005	9	0.0005	8
0.004..........	0.0014	25	0.0013	23	0.0012	20	0.0011	18	0.0010	16
0.006..........	0.0021	38	0.0020	34	0.0018	30	0.0016	27	0.0015	24
0.008..........	0.0029	50	0.0026	45	0.0024	40	0.0022	36	0.0020	31
0.010..........	0.0036	63	0.0033	57	0.0030	51	0.0027	45	0.0024	39
0.012..........	0.0043	75	0.0039	68	0.0036	61	0.0033	54	0.0029	47
0.014..........	0.0050	88	0.0046	79	0.0042	71	0.0038	63	0.0034	55
0.016..........	0.0057	100	0.0053	91	0.0048	81	0.0044	72	0.0039	63
0.018..........	0.0064	113	0.0059	102	0.0054	91	0.0049	81	0.0044	71
0.020..........	0.0071	126	0.0066	113	0.0060	101	0.0055	90	0.0049	78
0.022..........	0.0079	138	0.0072	125	0.0066	111	0.0060	99	0.0054	86
0.024..........	0.0086	151	0.0079	136	0.0072	121	0.0066	108	0.0059	94
0.026..........	0.0093	163	0.0086	147	0.0078	132	0.0071	116	0.0064	102
0.028..........	0.0100	176	0.0092	158	0.0084	142	0.0076	125	0.0069	110
0.030..........	0.0107	188	0.0099	170	0.0090	152	0.0082	134	0.0074	118

See instructions for use under Table 16.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

DIAGRAM 20.—CASE I—2000-LB. CONCRETE—n = 15

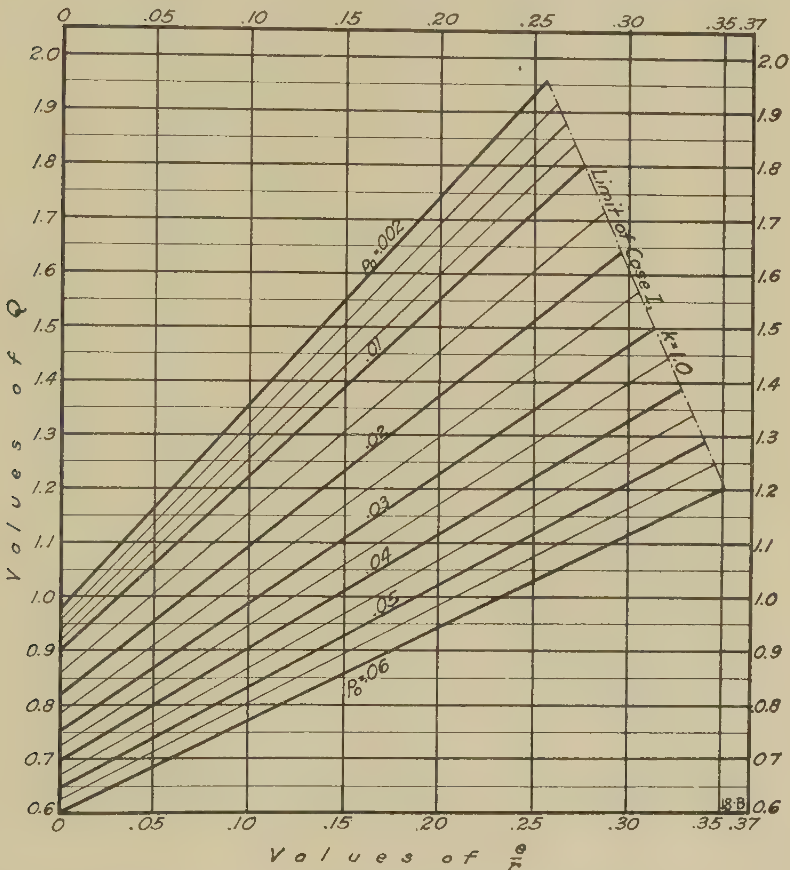


GENERAL NOTE FOR DIAGRAMS 20 TO 65

These diagrams are based on circular or rectangular sections in which the reinforcement is symmetrically placed with respect to gravity axis at right angles to the plane in which the load is eccentric.

INSTRUCTIONS FOR USE OF DIAGRAMS 20 TO 24.—Enter the diagram with the value of e/r and proceed vertically to an intersection with the sloping index line for an assumed value of p_0. From this intersection pass horizontally to the right or left marginal scale and read off the value of Q. Solve for f_c in formula (106).

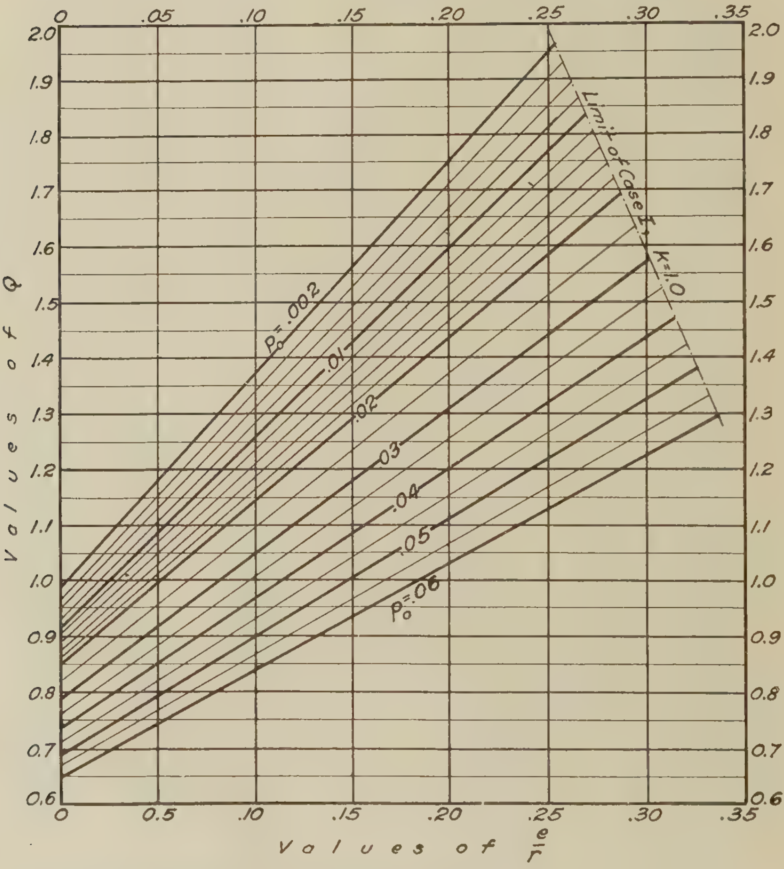
BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS
DIAGRAM 21.—CASE I—2500-LB. CONCRETE—$n = 12$



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

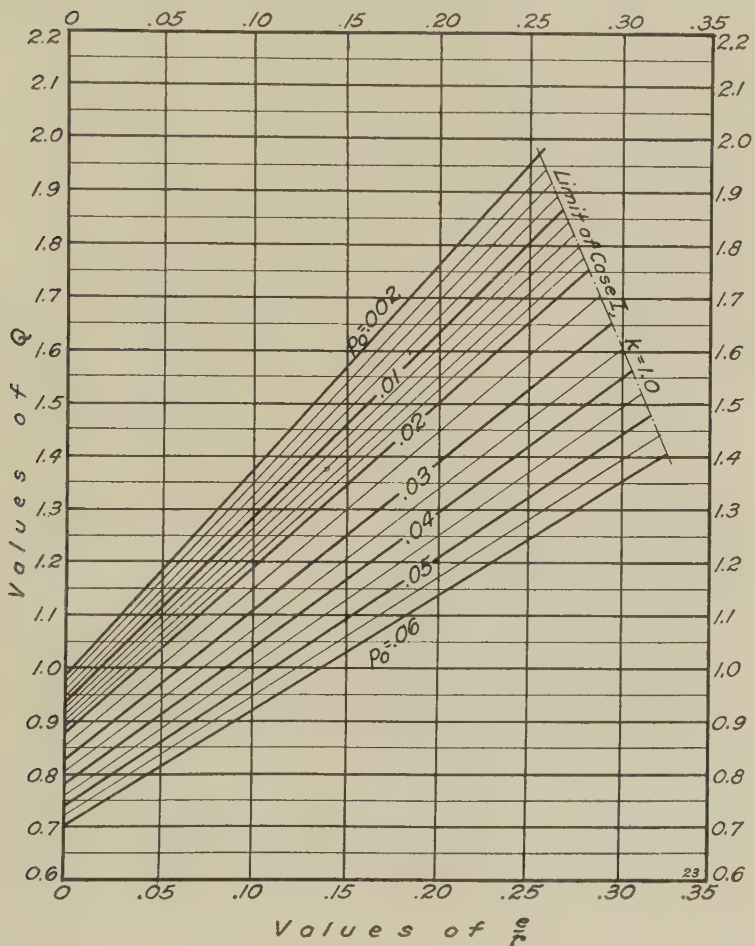
DIAGRAM 22.—CASE I—3000-LB. CONCRETE—n = 10



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

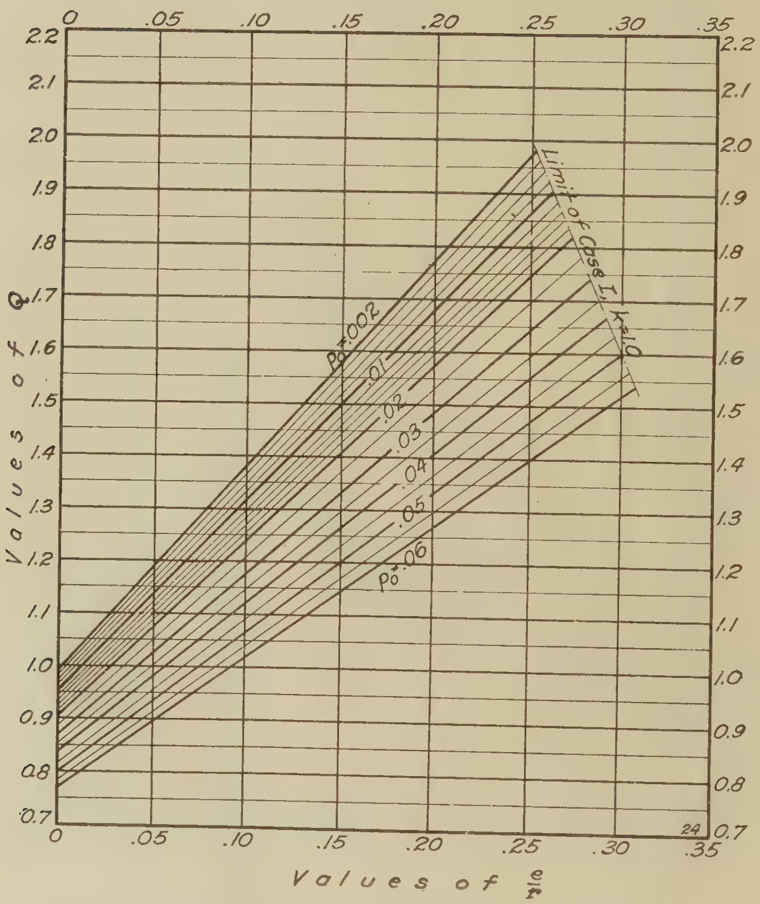
DIAGRAM 23.—CASE I—3750-LB. CONCRETE—$n = 8$



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

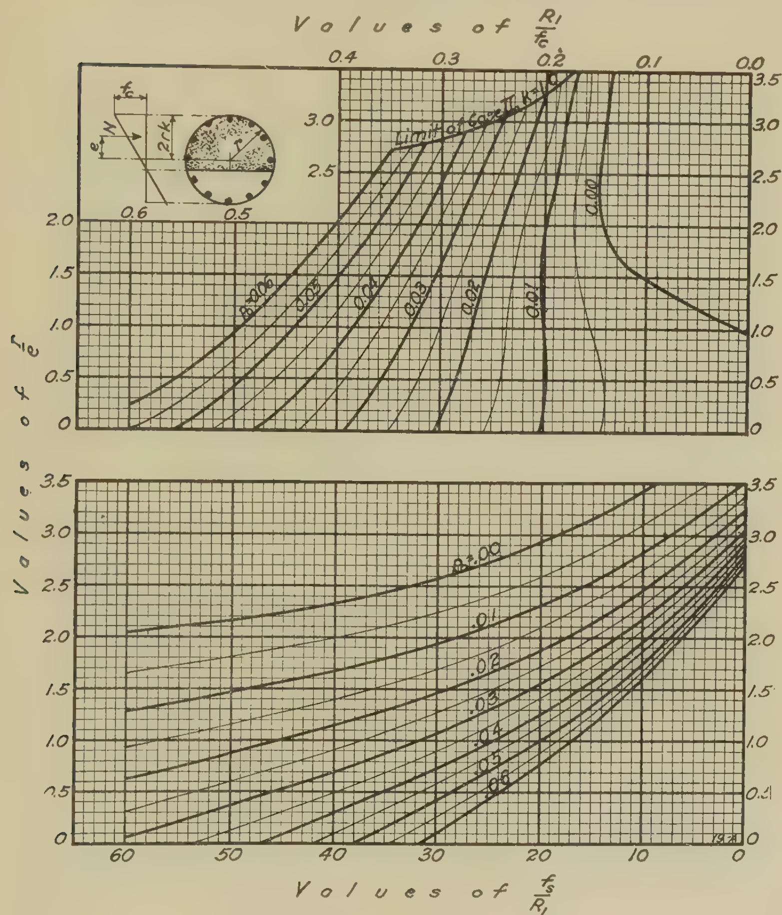
DIAGRAM 24.—CASE I—5000-LB. CONCRETE—$n = 6$



See instructions for use and also general note under Diagram 20.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

DIAGRAM 25.—CASE II—2000-LB. CONCRETE—$n = 15$



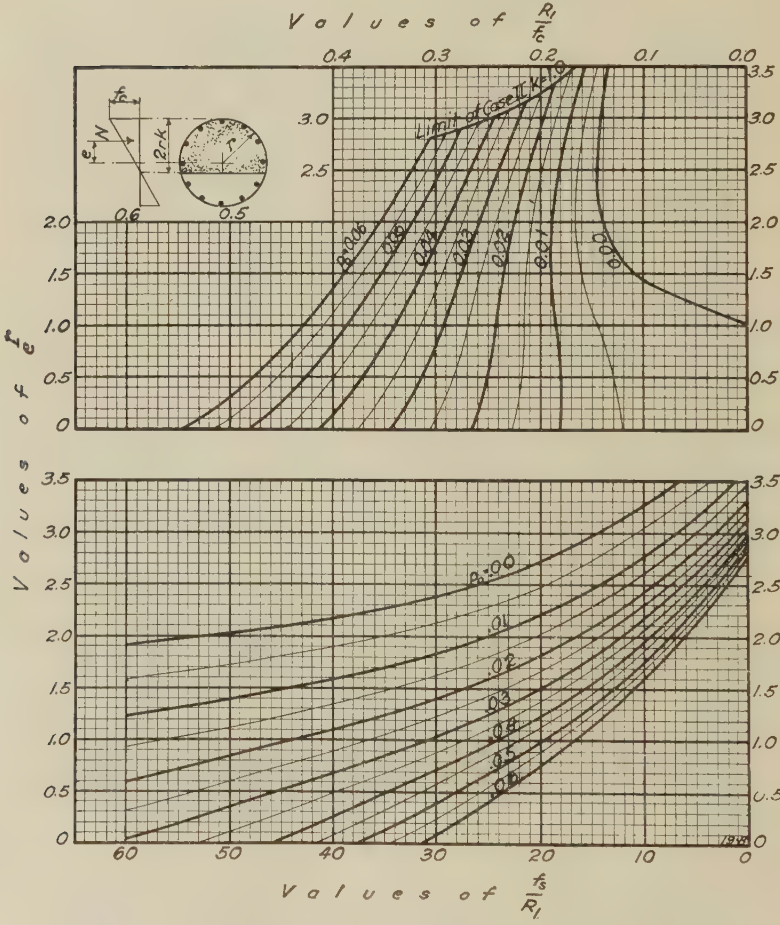
INSTRUCTIONS FOR USE OF DIAGRAMS 25 TO 29.—Enter the upper part of the diagram with the value of r/e and proceed horizontally to an intersection with the index line for an assumed value of p_o. Pass vertically to the upper marginal scale and read off the value R_1/fc.

Enter the lower part of the diagram in the same manner. From the intersection with the p_o index line pass vertically to the lower marginal scale and read off the value of fs/R_1.

Solve formulas (107), (108) and (109) for values of fc and fs. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

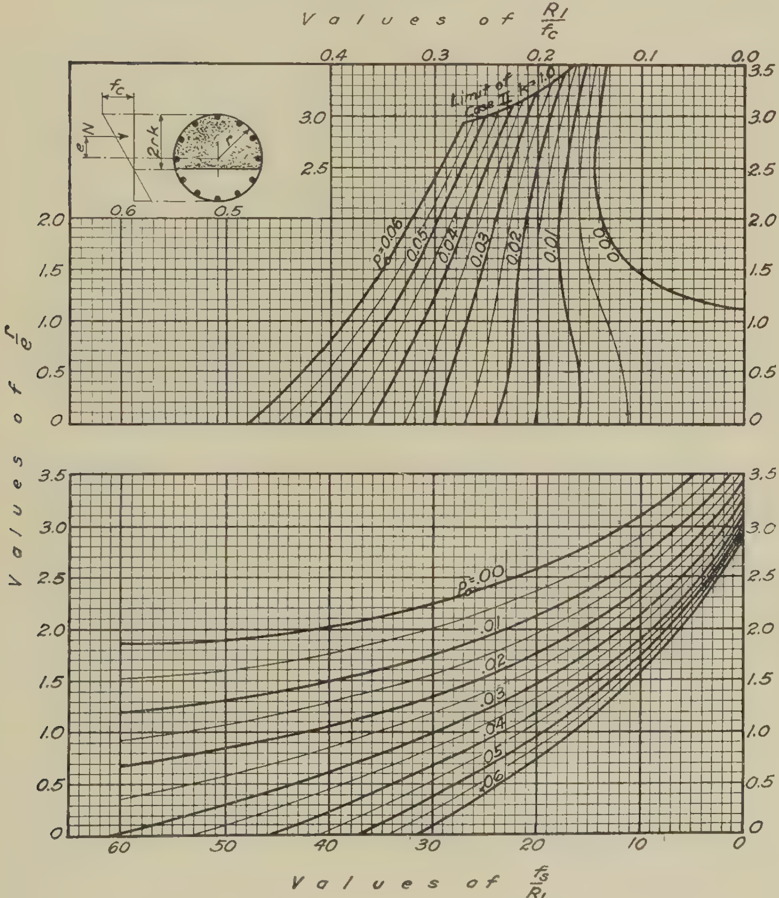
DIAGRAM 26.—CASE II—2500-LB. CONCRETE—$n = 12$



See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

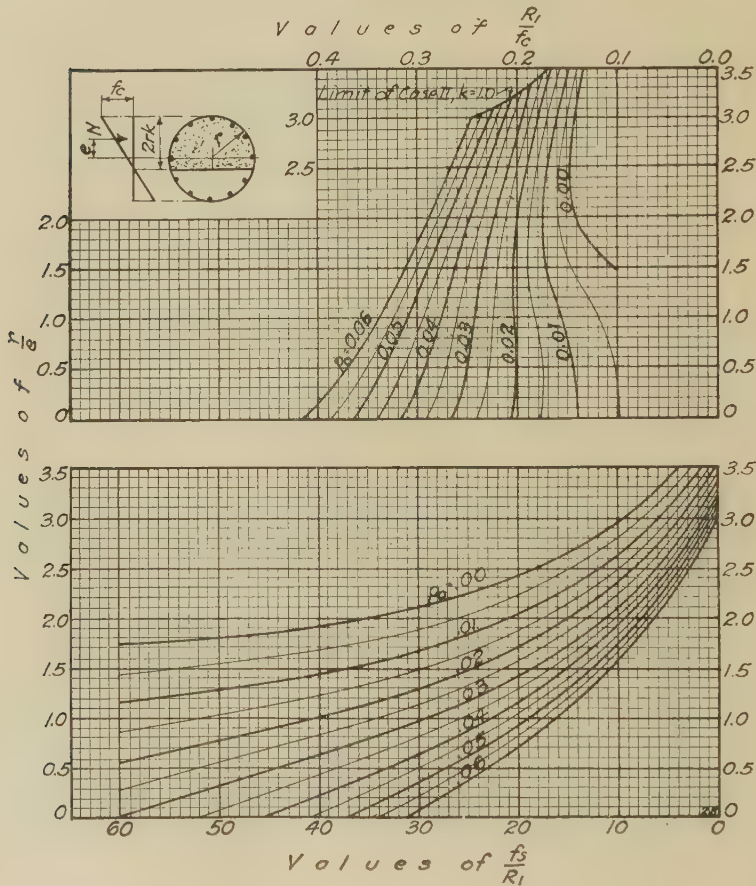
DIAGRAM 27.—CASE II—3000-LB. CONCRETE—$n = 10$



See instructions for use under Diagram 25.

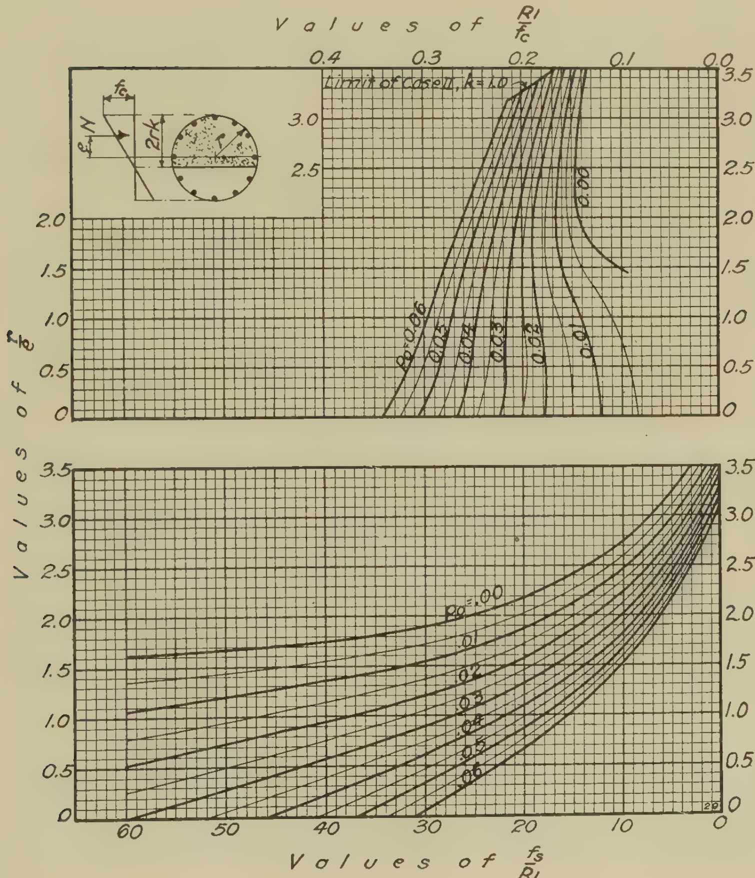
BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS

DIAGRAM 28.—CASE II—3750-LB. CONCRETE—n = 8



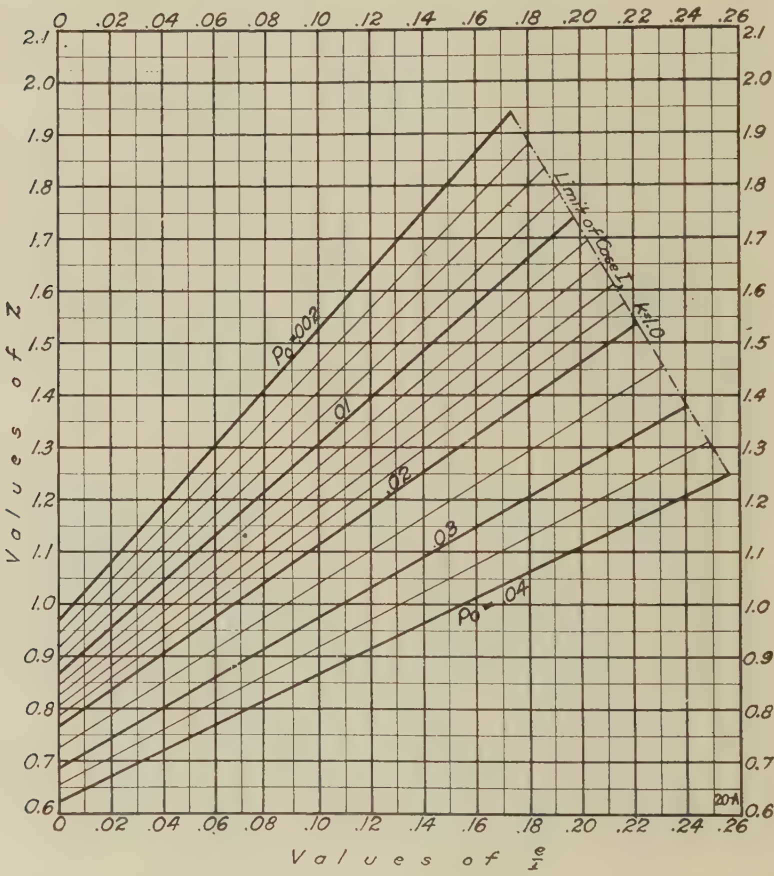
See instructions for use under Diagram 25.

BENDING AND DIRECT COMPRESSION—CIRCULAR SECTIONS
DIAGRAM 29.—CASE II—5000-LB. CONCRETE—$n = 6$



See instructions for use under Diagram 25.

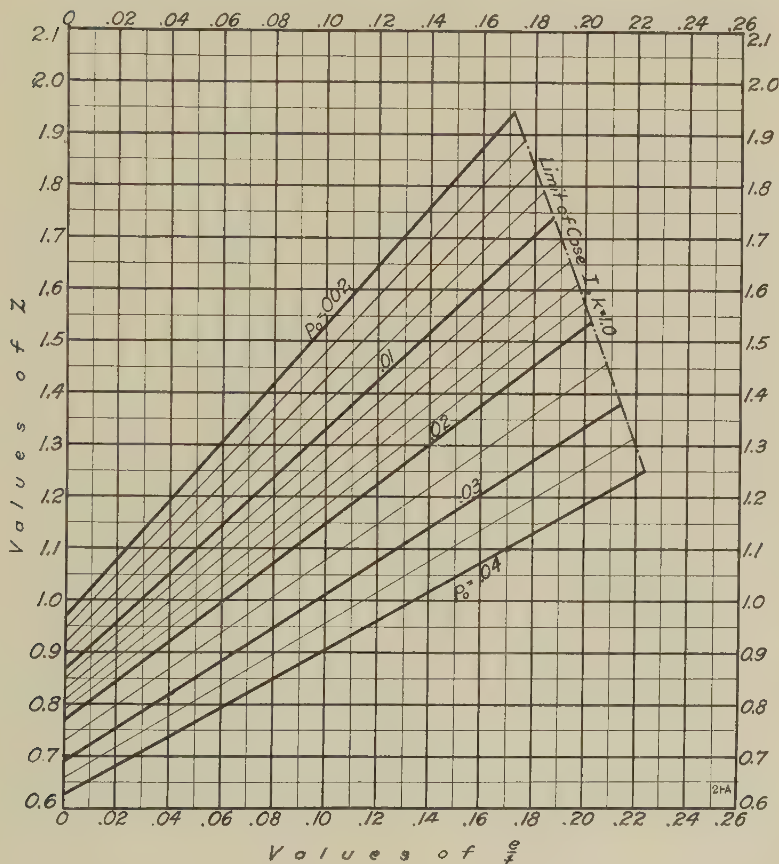
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 30.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.05t$



INSTRUCTIONS FOR USE OF DIAGRAMS 30 TO 45.—Enter the diagram with the value of e/t and proceed vertically to an intersection with the sloping index line for an assumed value of p_0. From this intersection pass horizontally to the right or left marginal scale and read off the value of Z. Solve formula (110) for the stress in the concrete. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

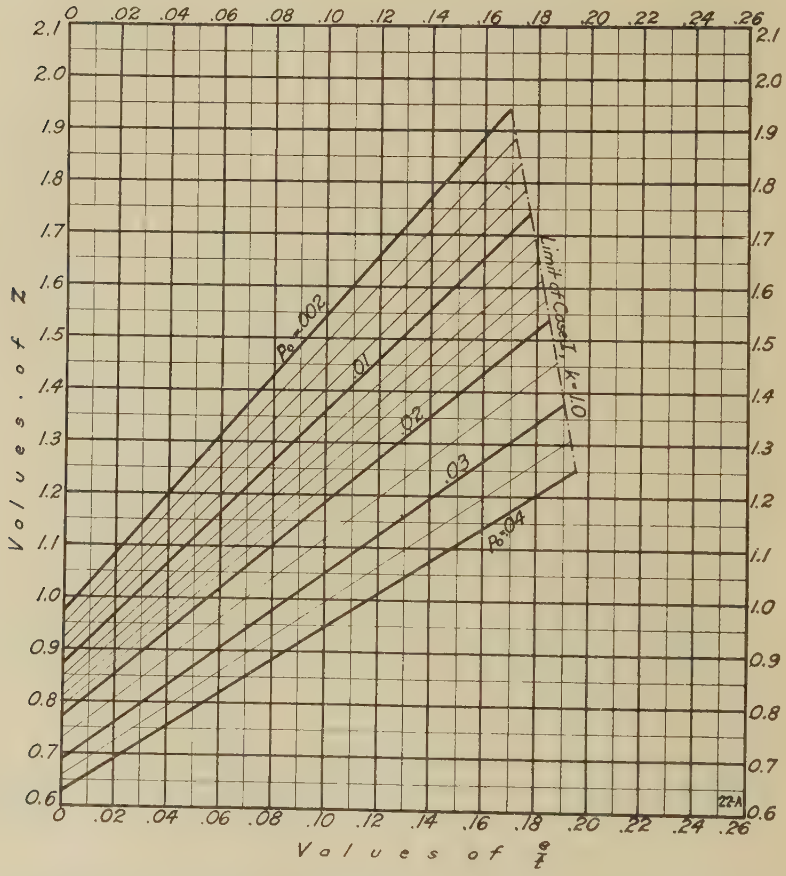
DIAGRAM 31.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.1t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

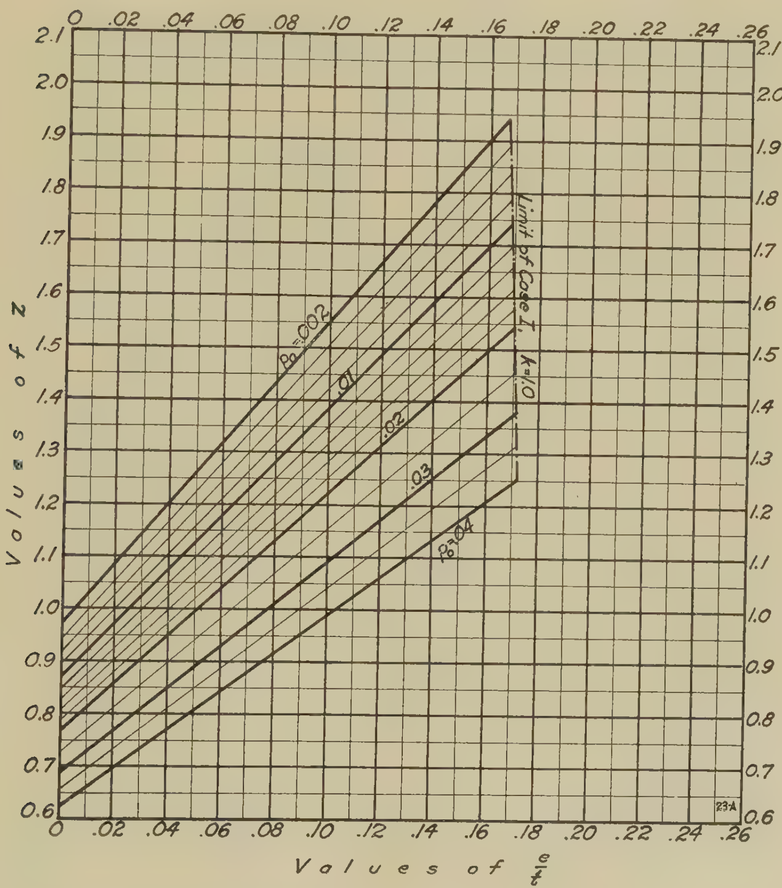
DIAGRAM 32.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.15t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

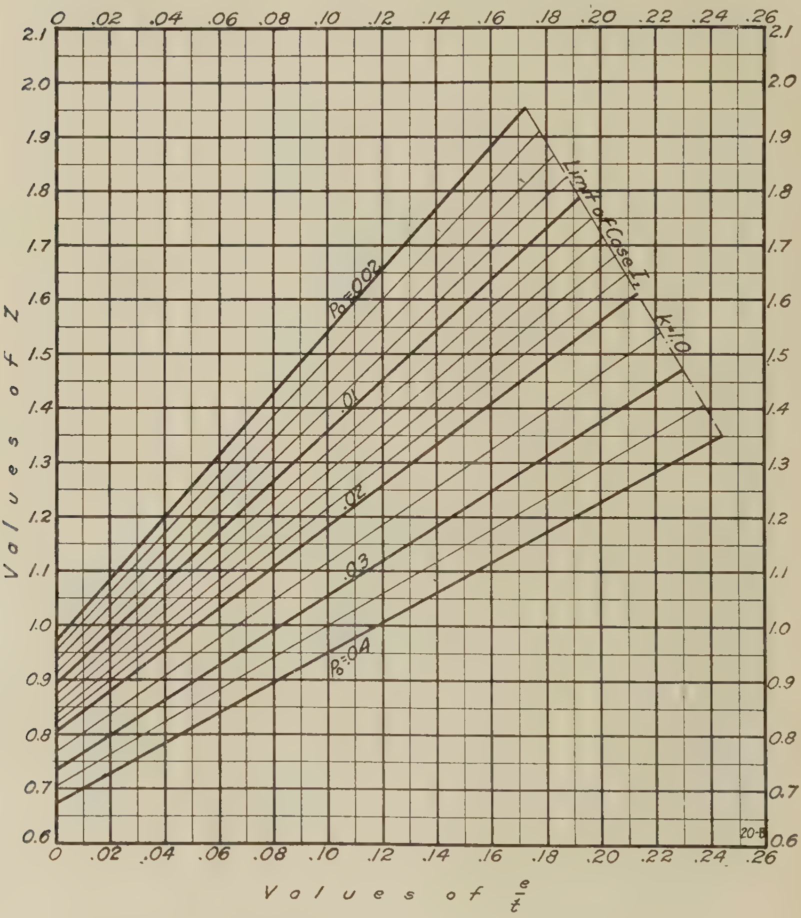
DIAGRAM 33.—CASE I—2000-LB. CONCRETE—$n = 15$—$d' = 0.2t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

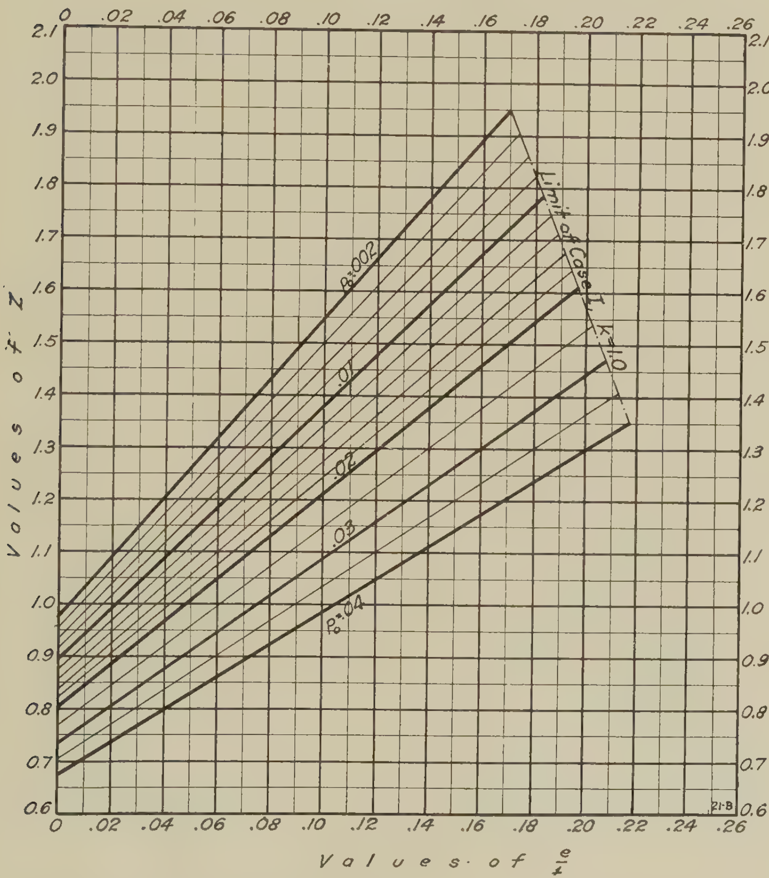
DIAGRAM 34.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.05t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

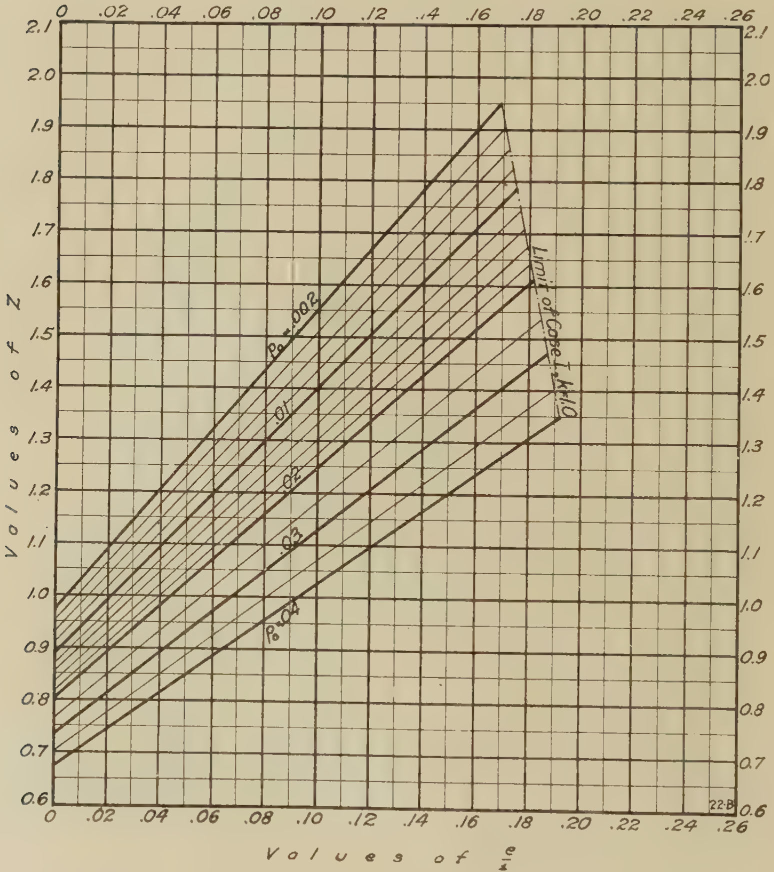
DIAGRAM 35.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.1t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

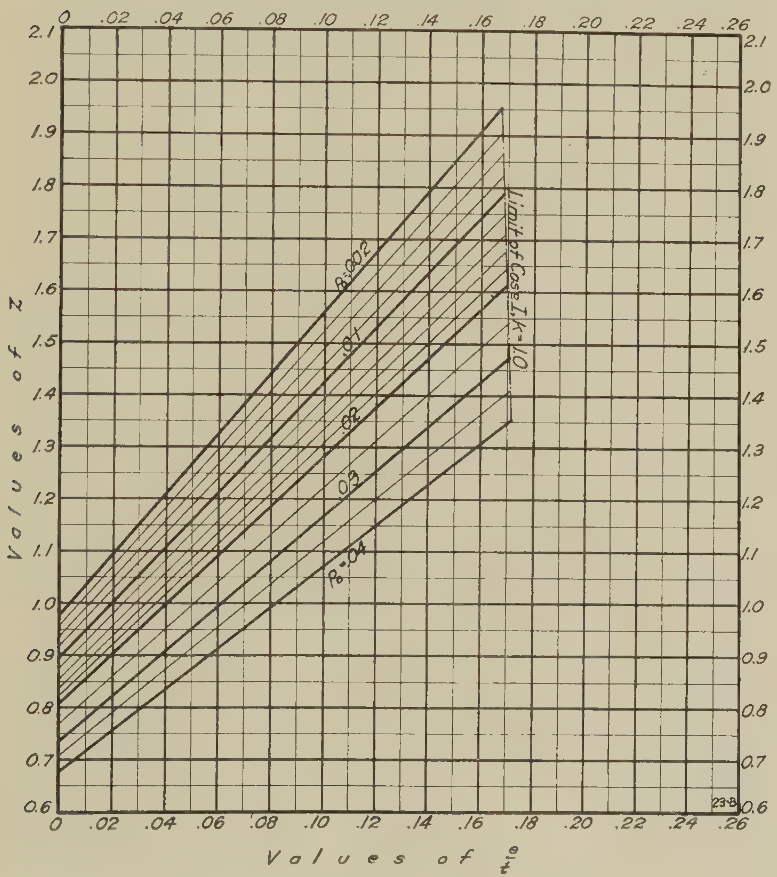
DIAGRAM 36.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.15t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

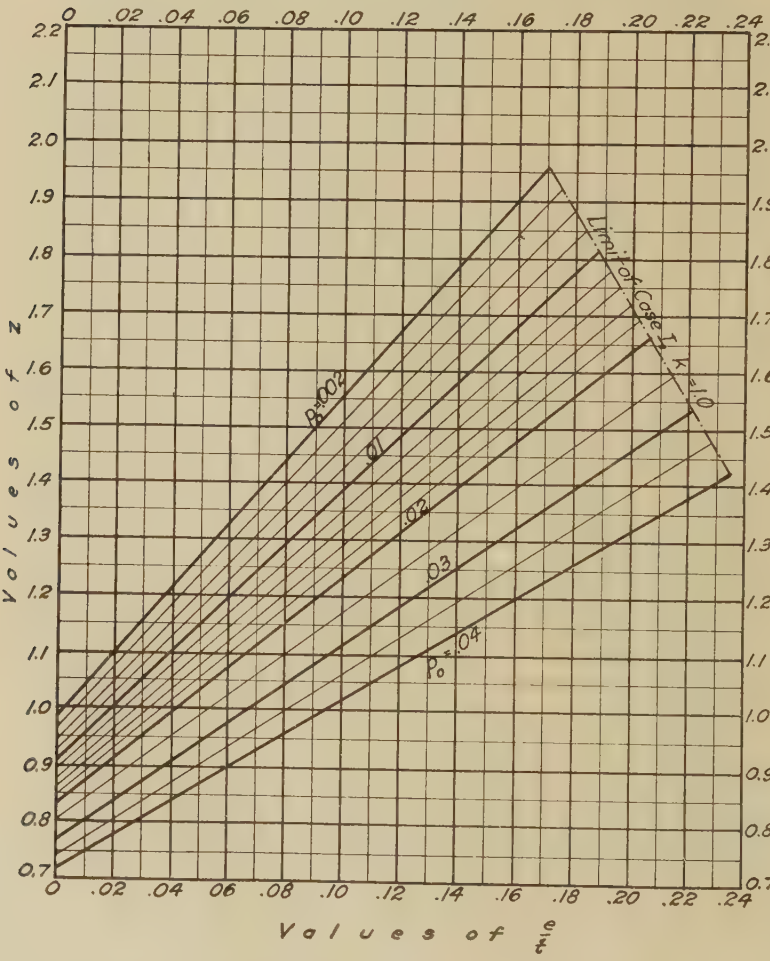
DIAGRAM 37.—CASE I—2500-LB. CONCRETE—$n = 12$—$d' = 0.2t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

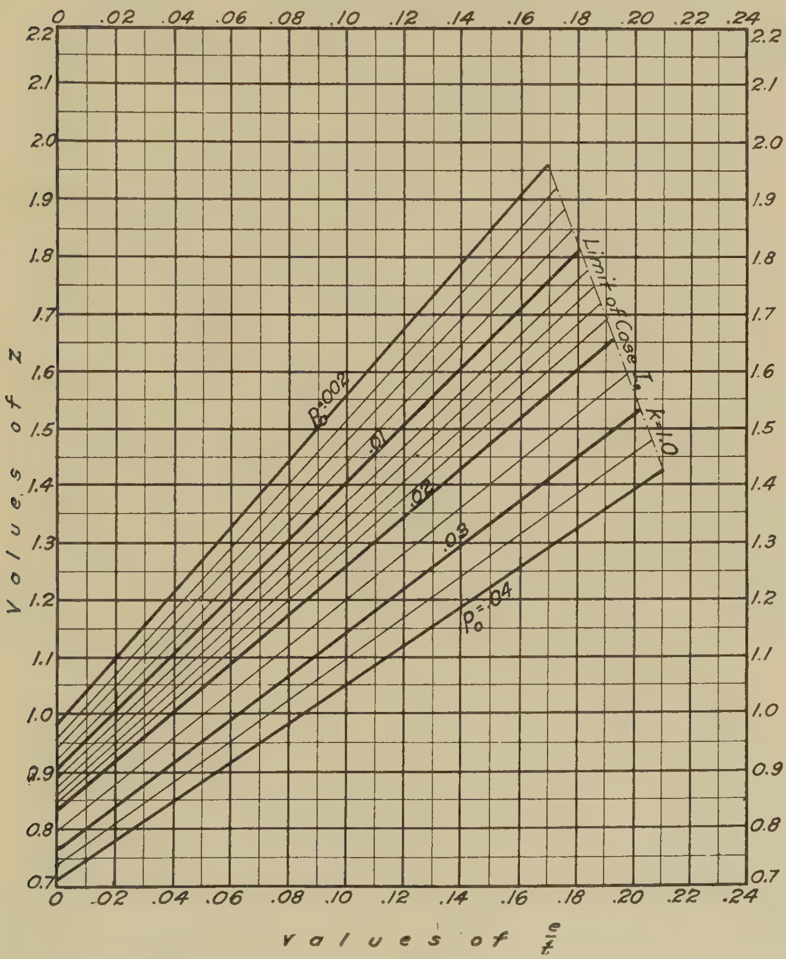
DIAGRAM 38.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.05t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

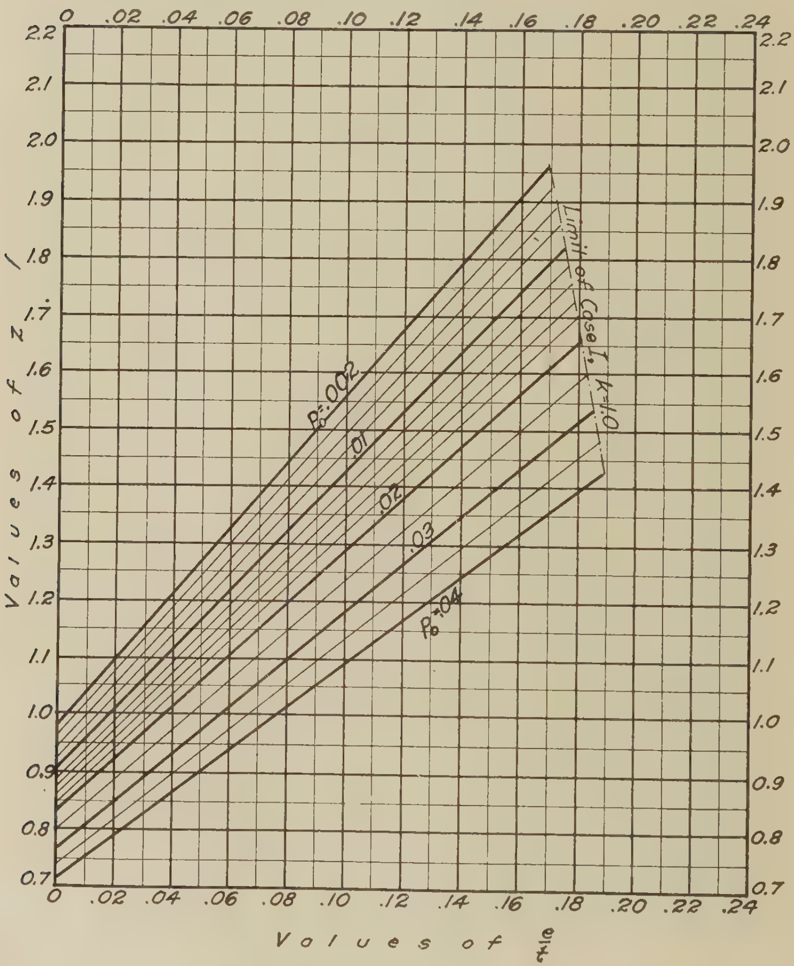
DIAGRAM 39.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.1t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

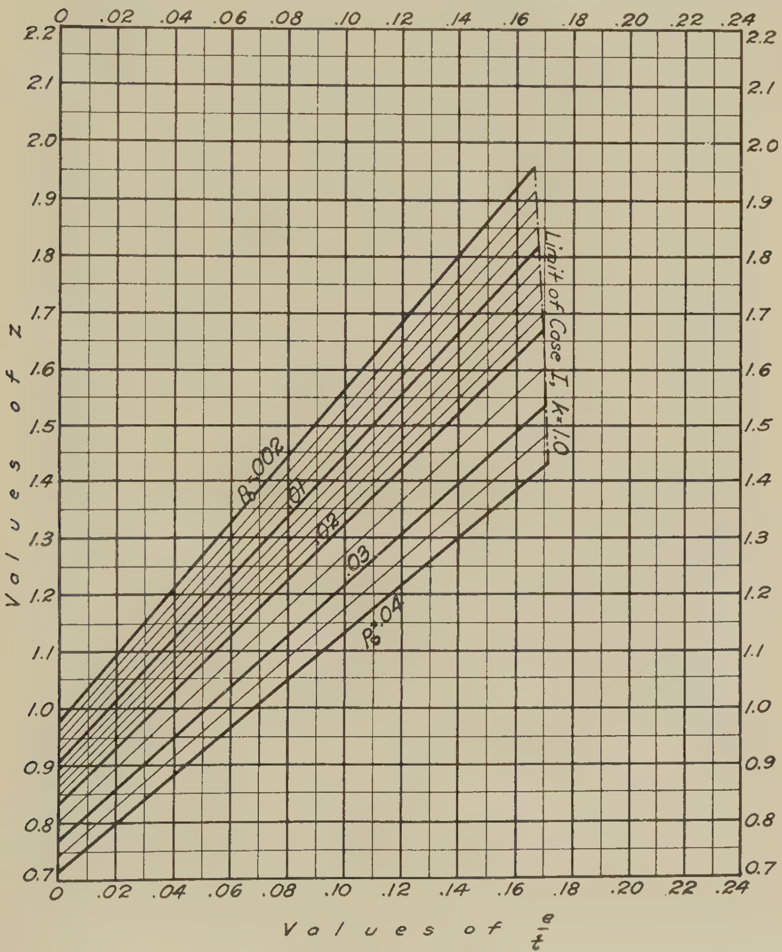
DIAGRAM 40.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.15t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

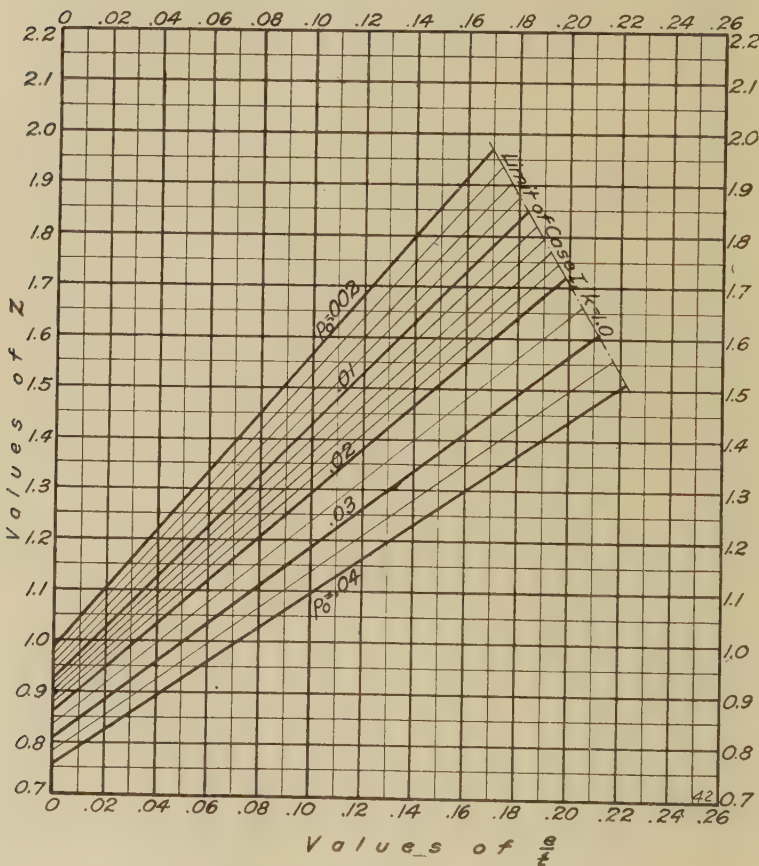
DIAGRAM 41.—CASE I—3000-LB. CONCRETE—$n = 10$ $d' = 0.2t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

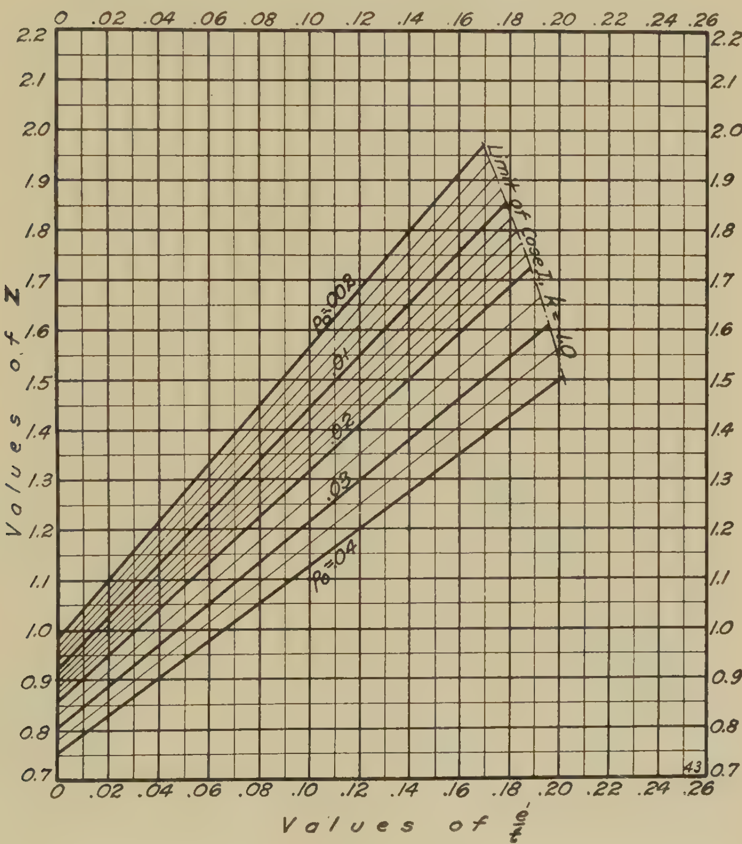
DIAGRAM 42.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.05t$



See instructions for use under Diagram 30, page 602.

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

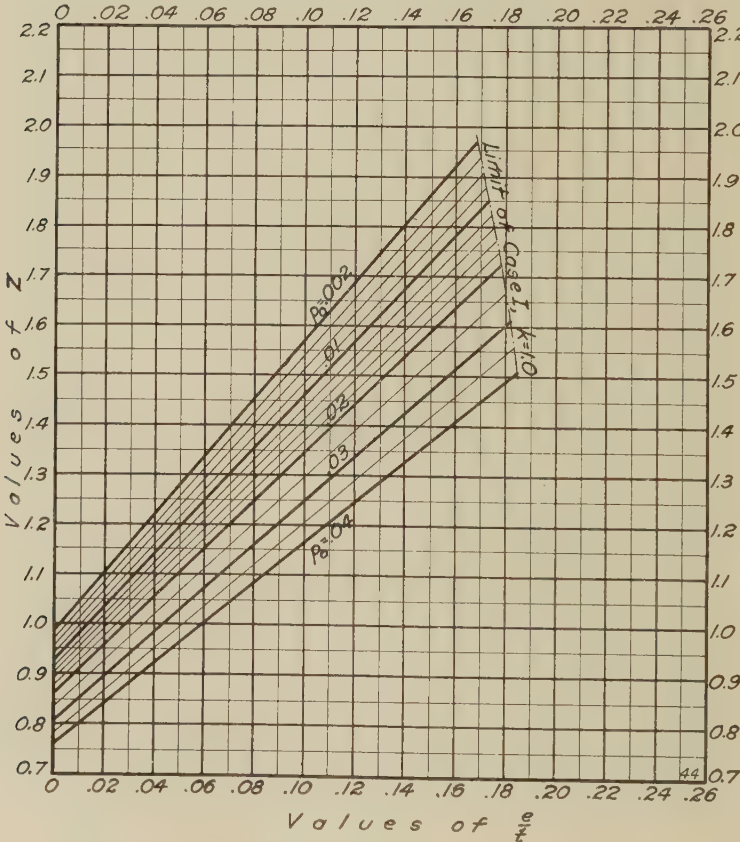
DIAGRAM 43.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.1t$



See instructions for use under Diagram 30, page 602.

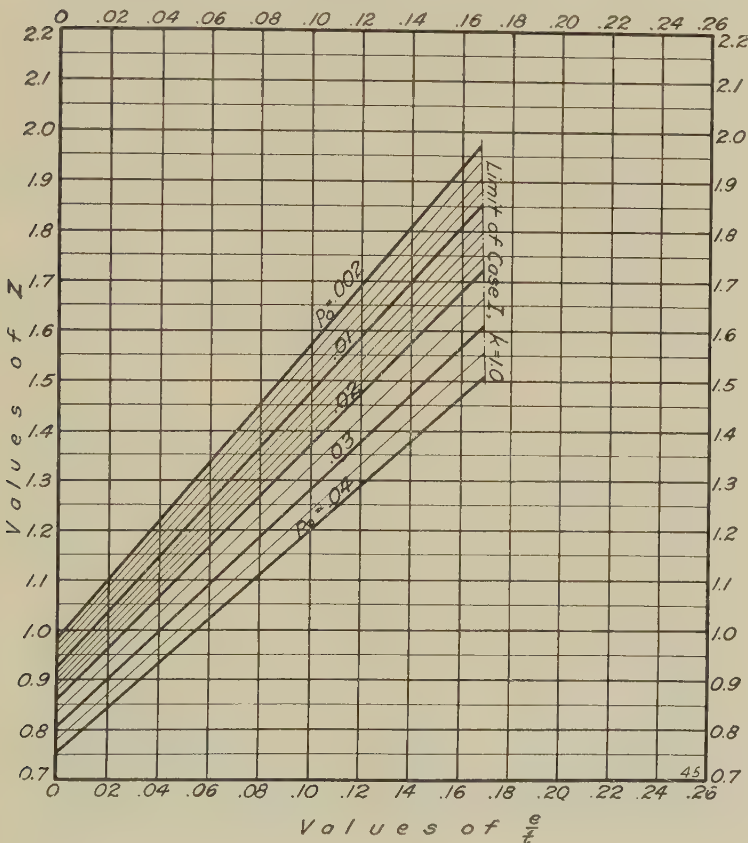
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 44.—CASE I—3750-LB. CONCRETE —$n = 8$—$d' = 0.15t$



See instructions for use under Diagram 30, page 602.

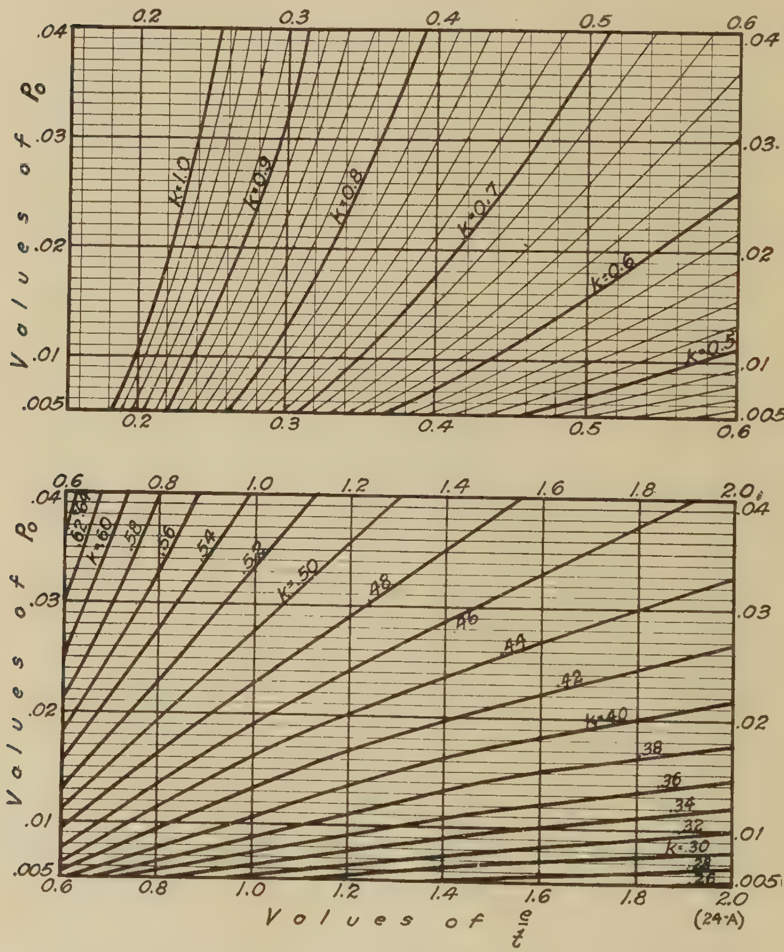
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 45.—CASE I—3750-LB. CONCRETE—$n = 8$—$d' = 0.2t$



See instructions for use under Diagram 30, page 602

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

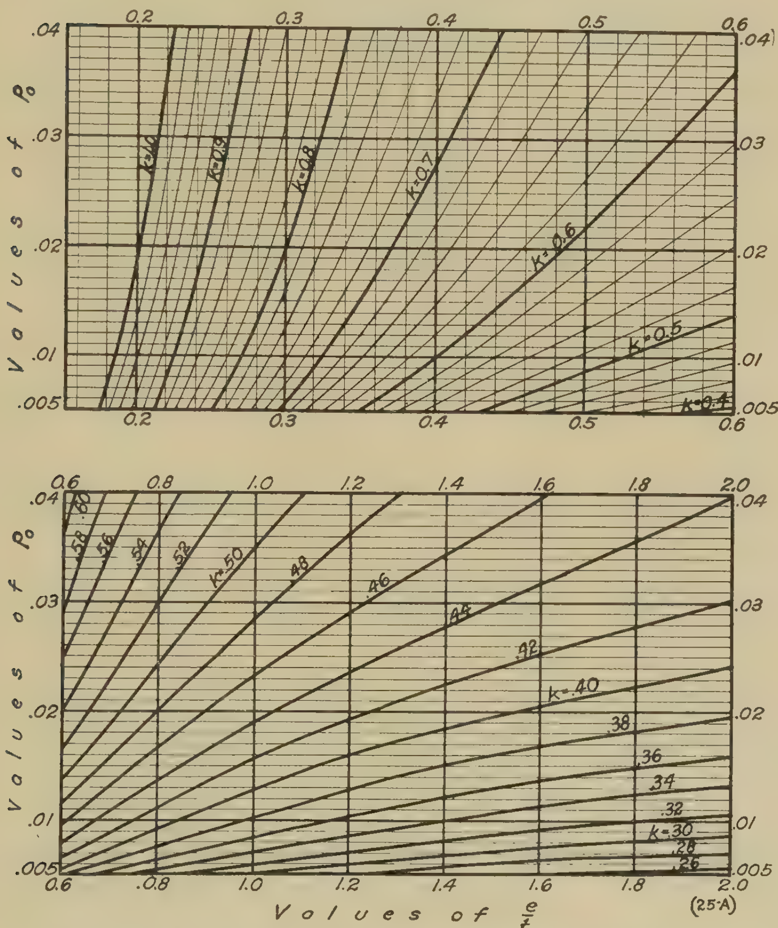
DIAGRAM 46.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.05t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersect on with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

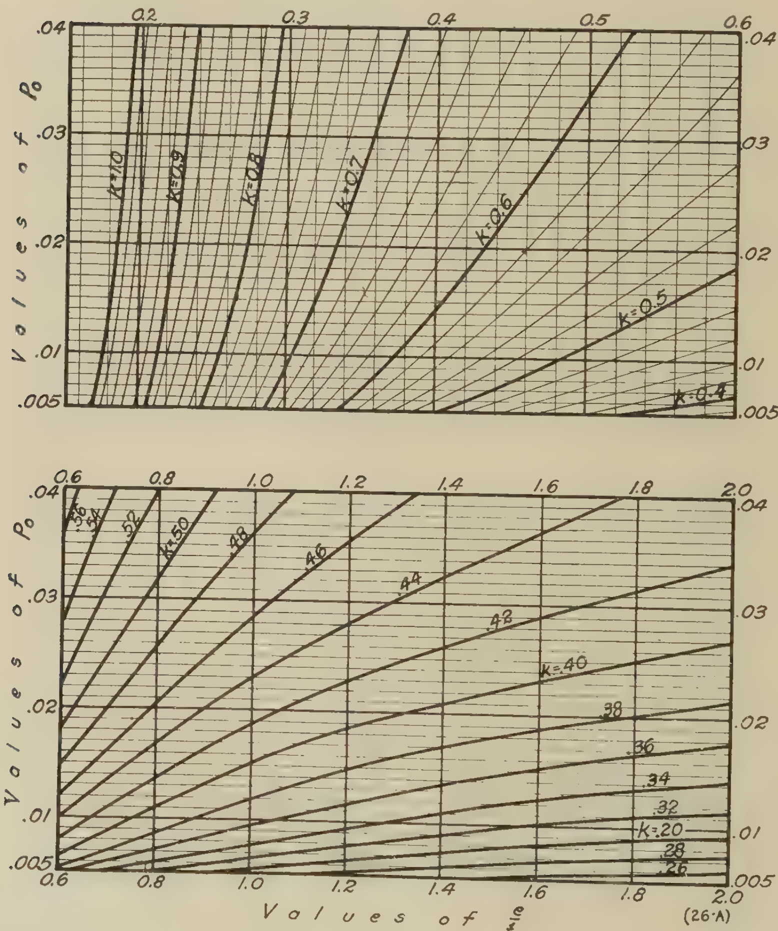
DIAGRAM 47.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.1t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

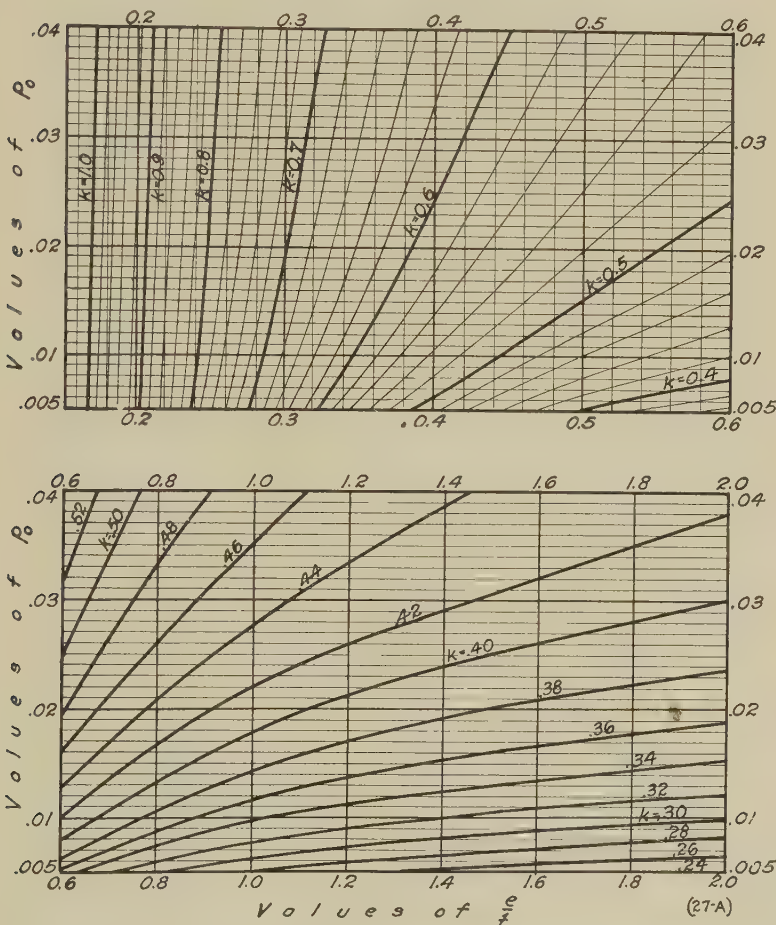
DIAGRAM 48.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

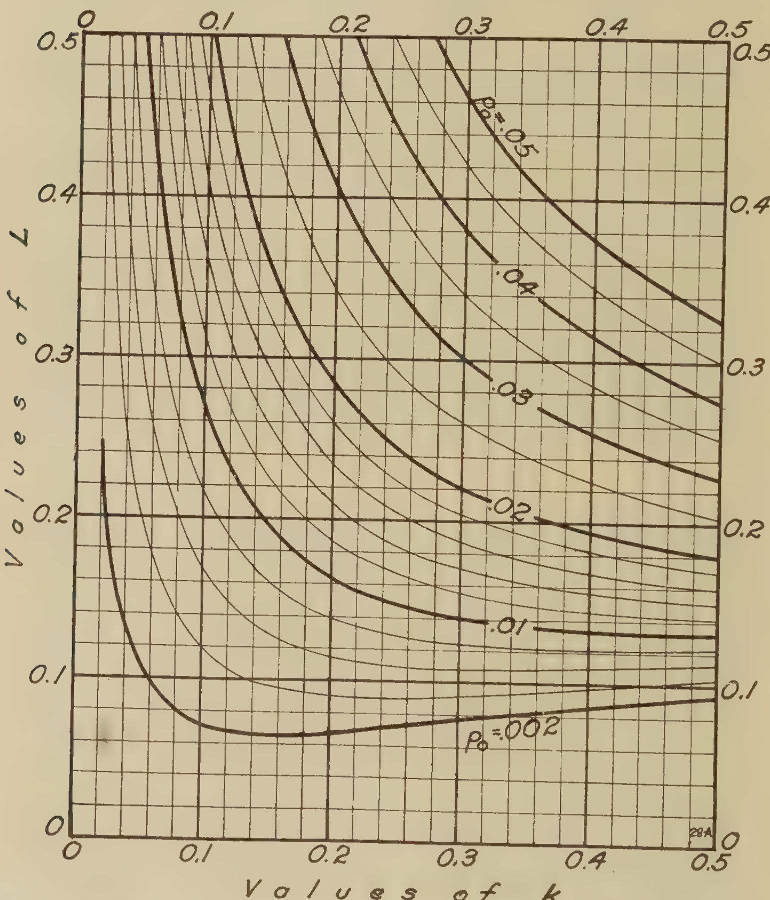
DIAGRAM 49.—CASE II—2000-LB. CONCRETE—$n = 15$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 50. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 50.—(PART ONE)—CASE II—2000-LB. CONCRETE—n = 15



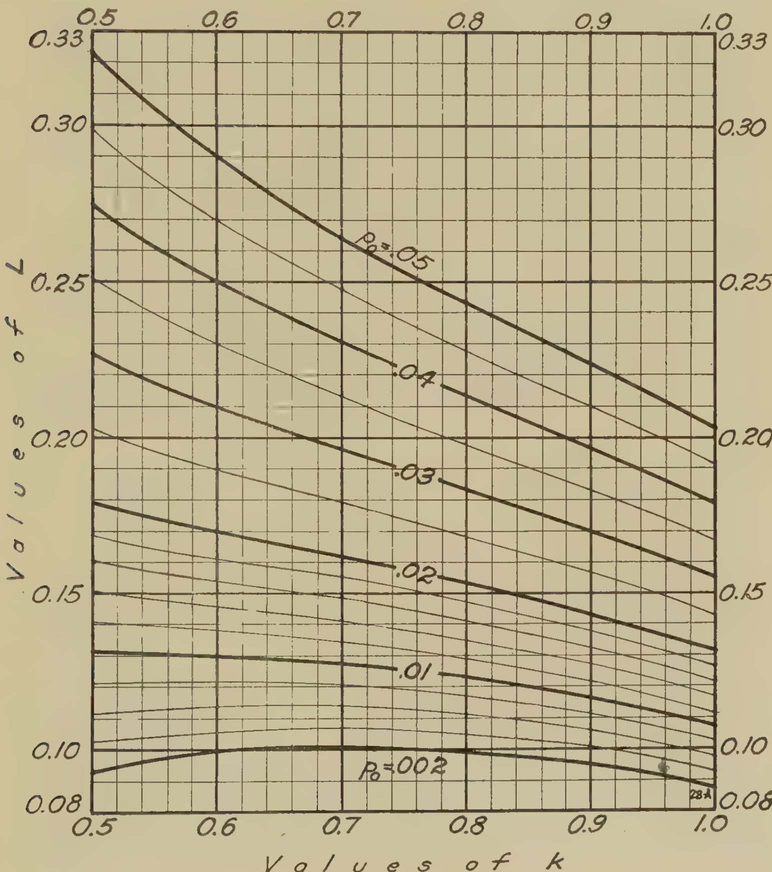
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 46 (or 47, 48 or 49, according to value of d'/t in the member being designed). The value of p_o used in those diagrams must be modified as follows:

p_o used in Diagram 46 must be divided by 0.79 ($d' = 0.05t$)
p_o used in Diagram 47 used without modification ($d' = 0.1t$)
p_o used in Diagram 48 must be divided by 1.306 ($d' = 0.15t$)
p_o used in Diagram 49 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two).

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 50.—(PART TWO)—CASE II—2000-LB. CONCRETE—$n = 15$

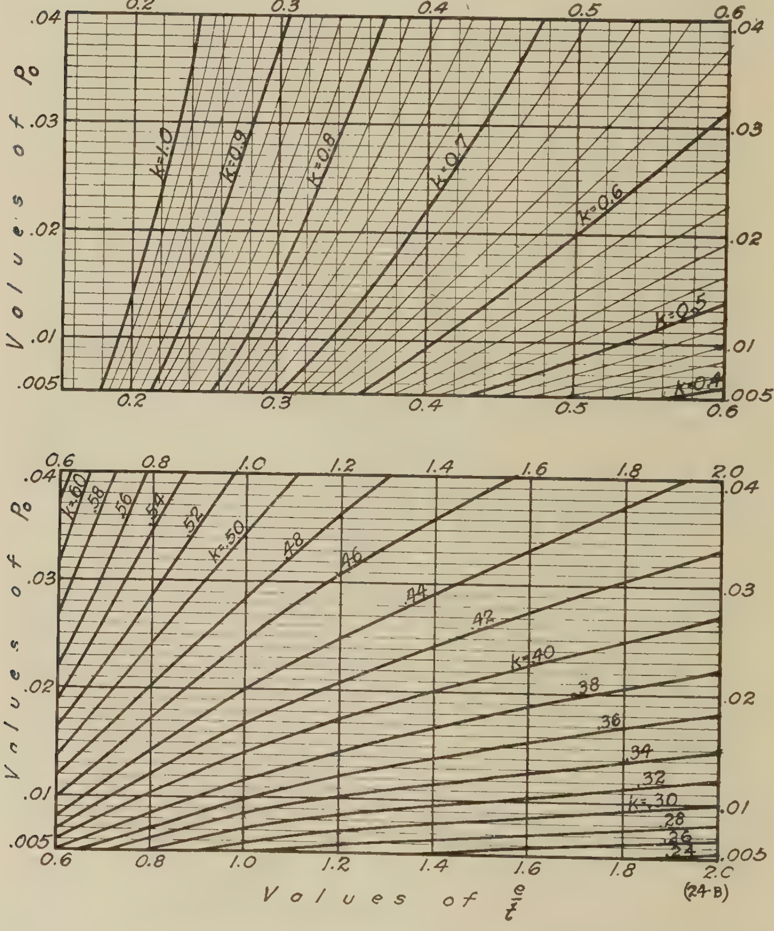


(Instructions continued from preceding page).

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_0. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

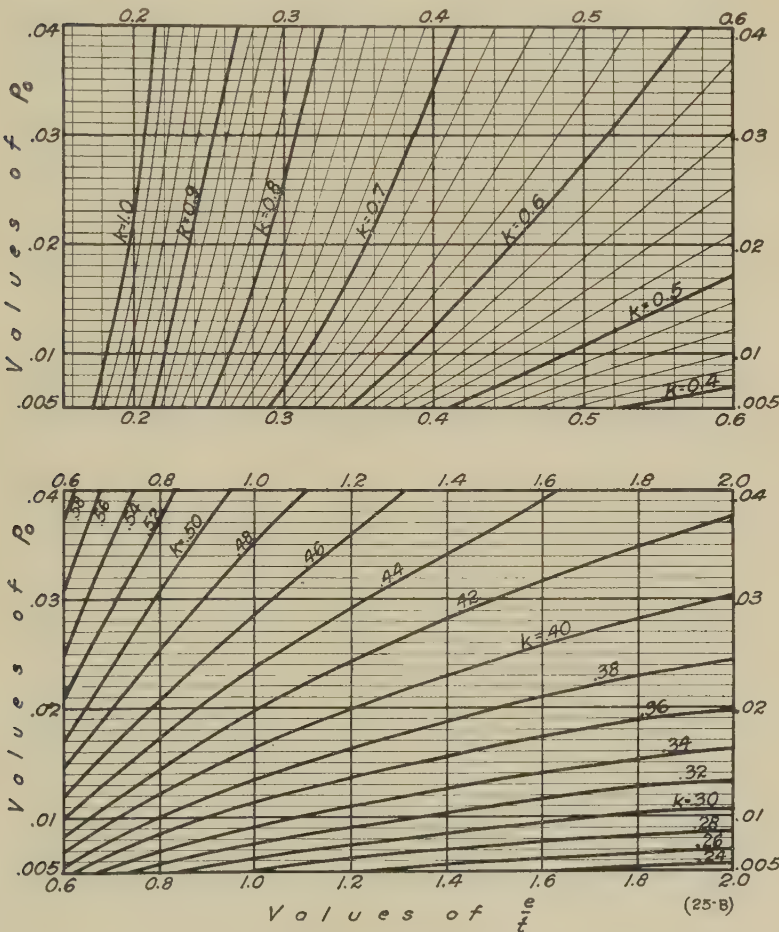
DIAGRAM 51.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.05t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

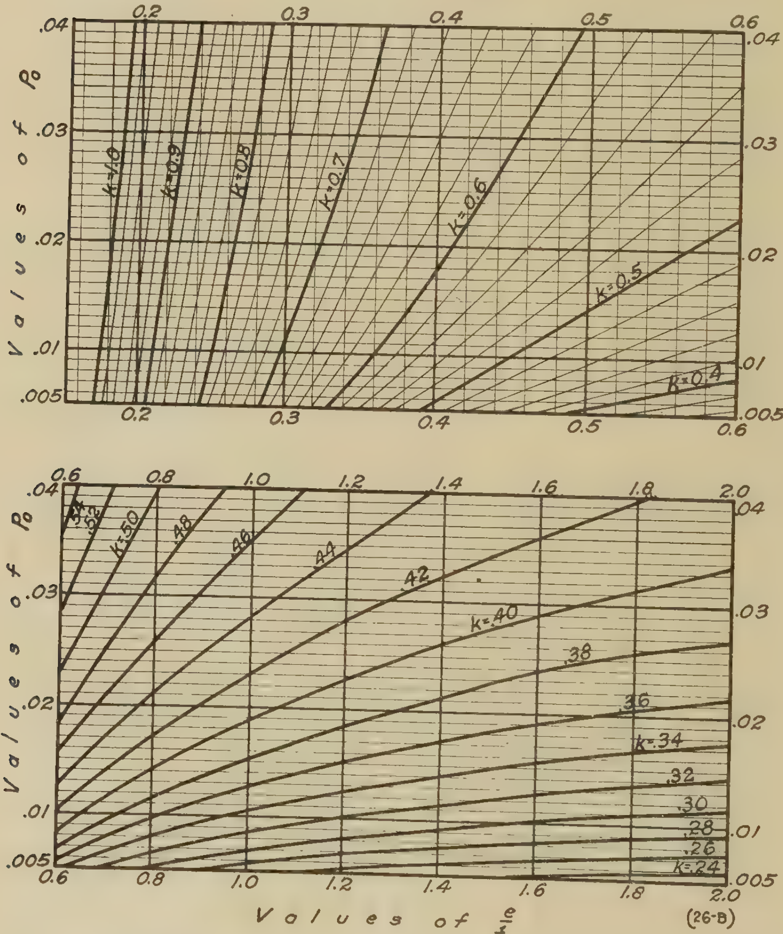
DIAGRAM 52.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.1t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

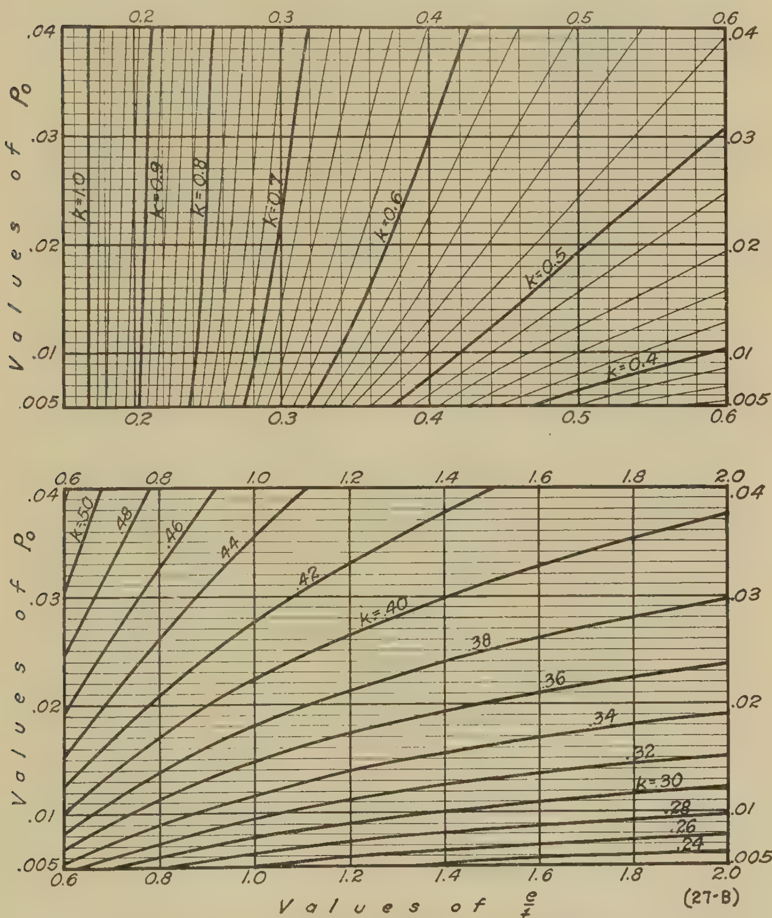
DIAGRAM 53.—CASE II—2500-LB CONCRETE—$n = 12$—$d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

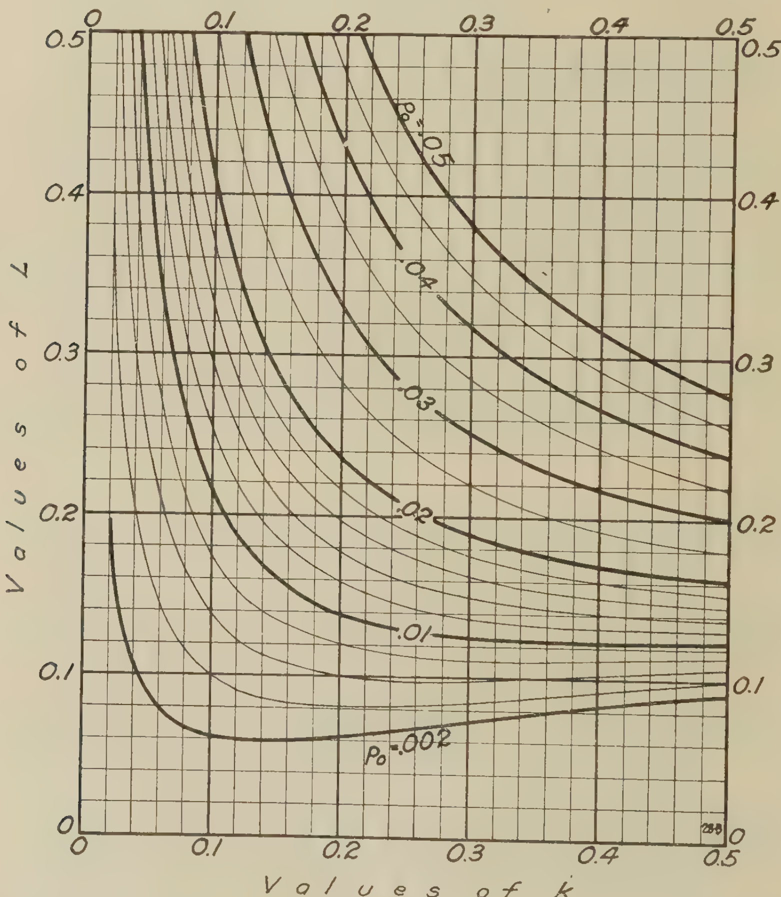
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 54.—CASE II—2500-LB. CONCRETE—$n = 12$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 55. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 55.—(PART ONE)—CASE II—2500-LB. CONCRETE—$n = 12$



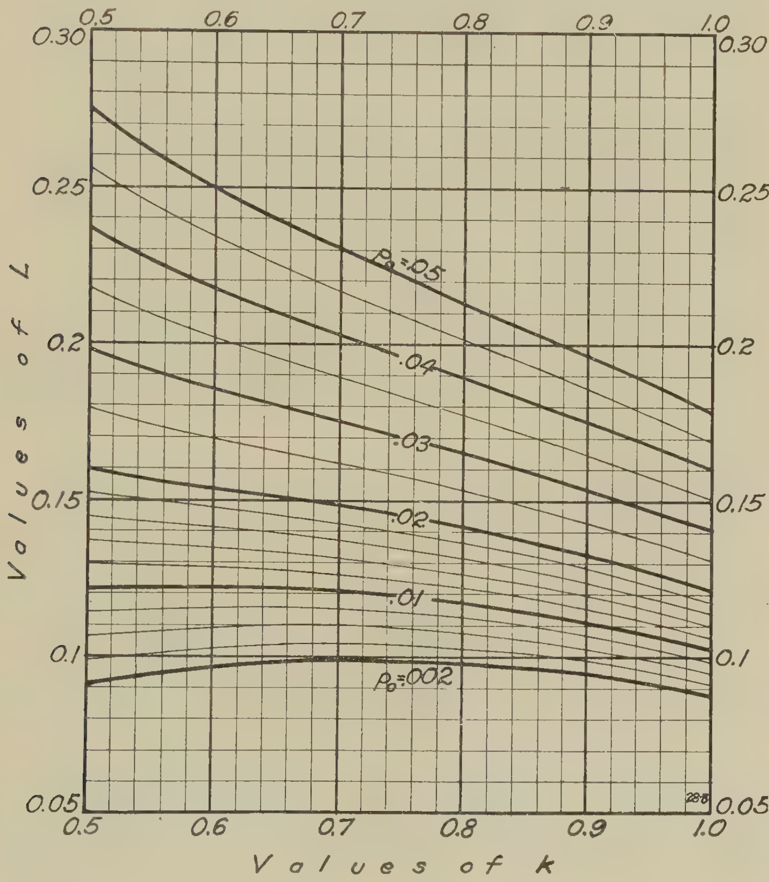
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 51 (or 52, 53, or 54, according to value of d'/t in the member being designed). The value of p_o used in those diagrams must be modified as follows:

p_o used in Diagram 51 must be divided by 0.79 ($d' = 0.05t$)
p_o used in Diagram 52 used without modification ($d' = 0.1t$)
p_o used in Diagram 53 must be divided by 1.306 ($d' = 0.15t$)
p_o used in Diagram 54 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 55.—(PART TWO)—CASE II—2500-LB. CONCRETE—$n = 12$

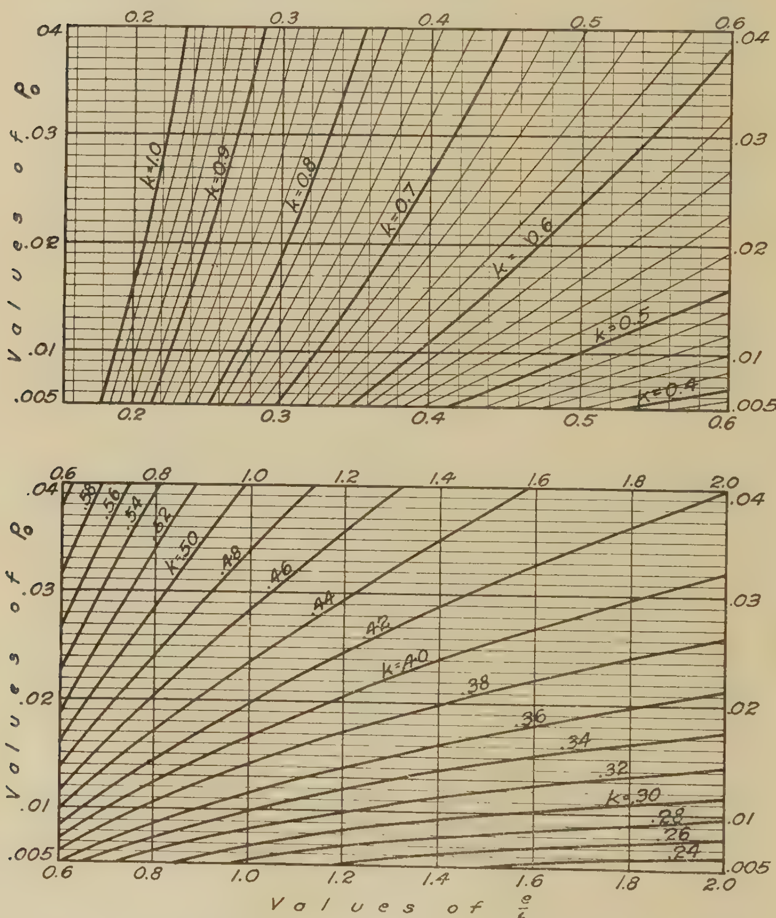


(Instructions continued from preceding page)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_0. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

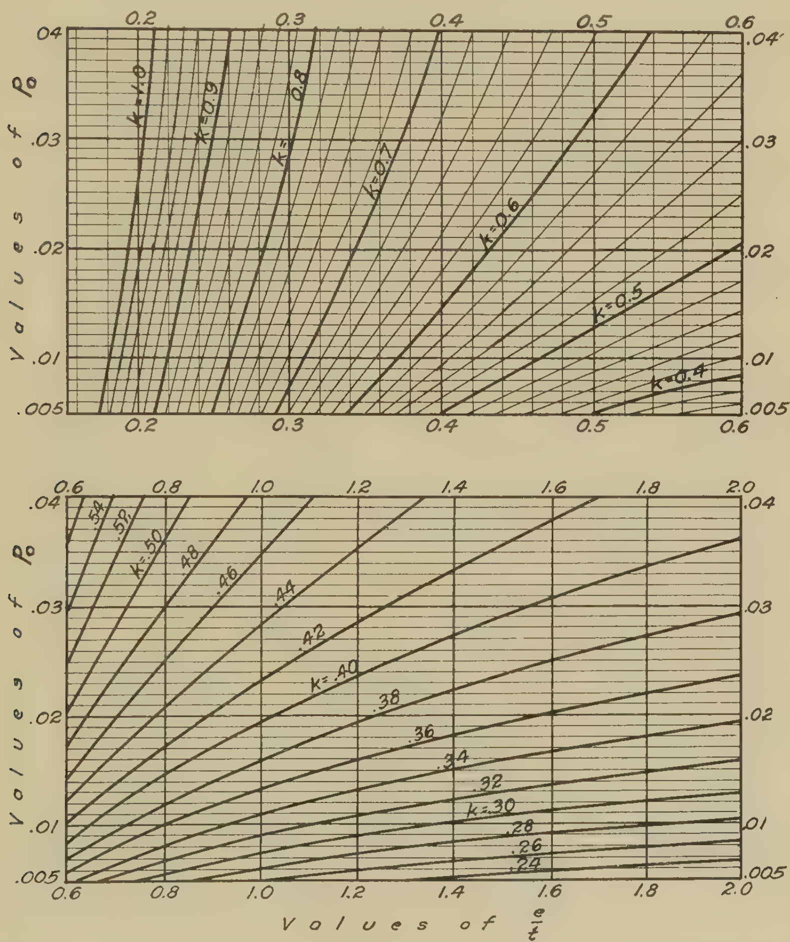
DIAGRAM 56.—CASE II—3000-LB. CONCRETE—$n = 10 \quad d' = 0.05t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

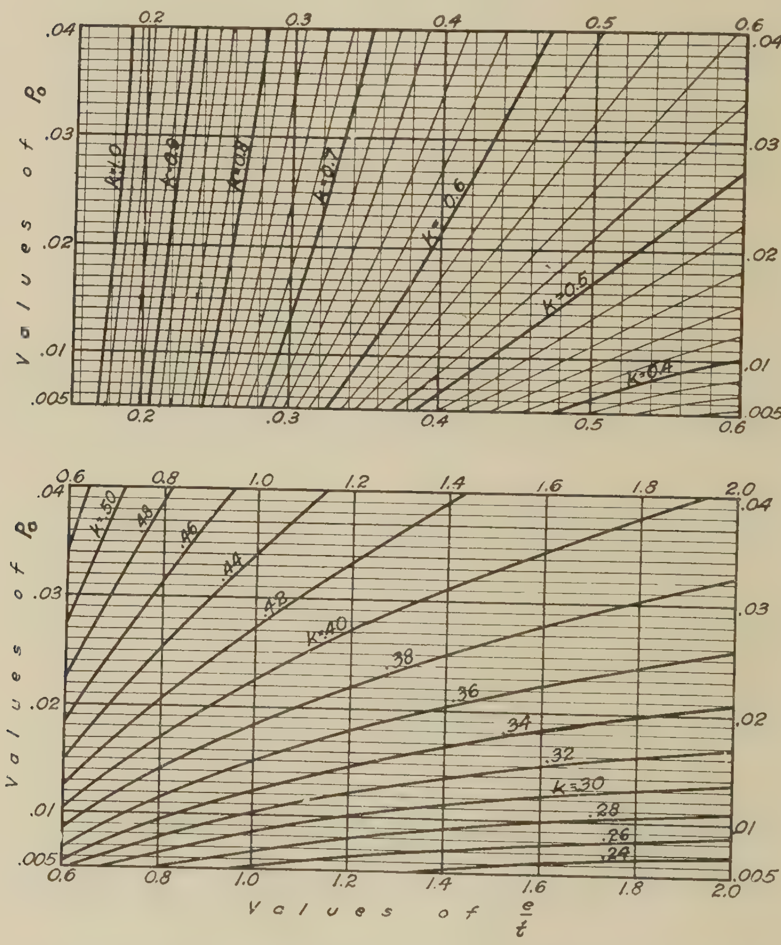
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 57.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.1t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

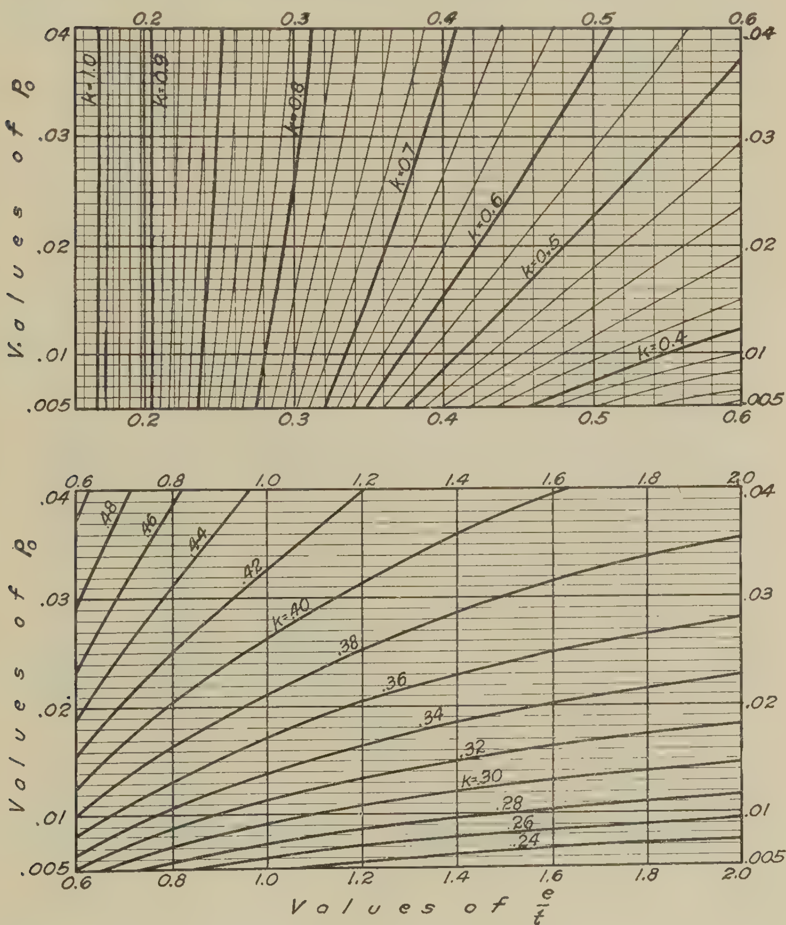
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 58.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

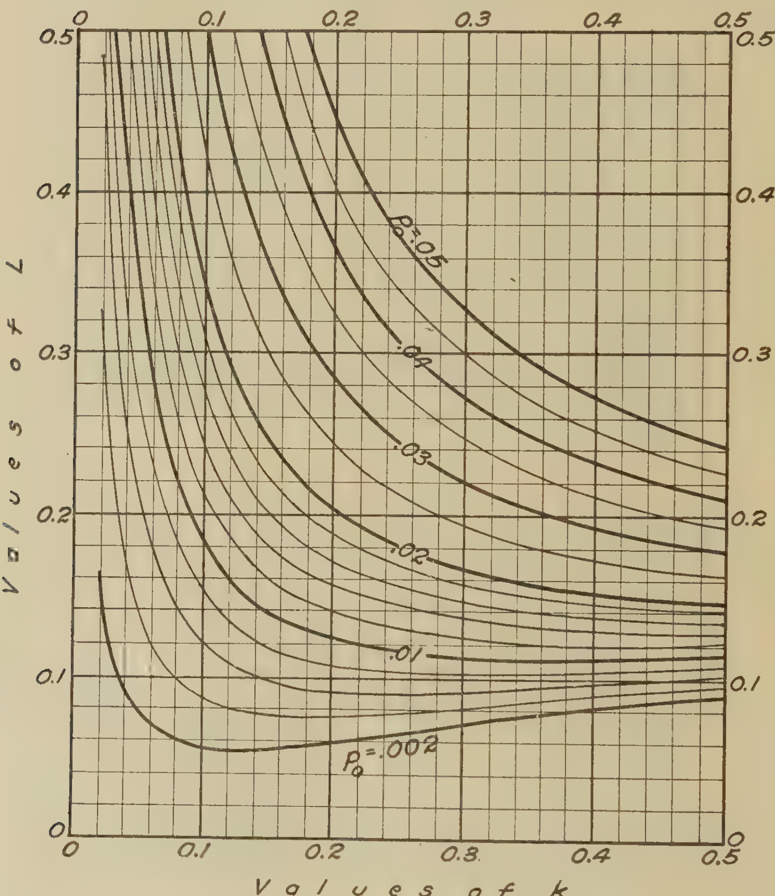
DIAGRAM 59.—CASE II—3000-LB. CONCRETE—$n = 10$ $d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the value of k. With this value of k enter Diagram 60. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 60.—(PART ONE)—CASE II—3000-LB. CONCRETE—$n = 10$



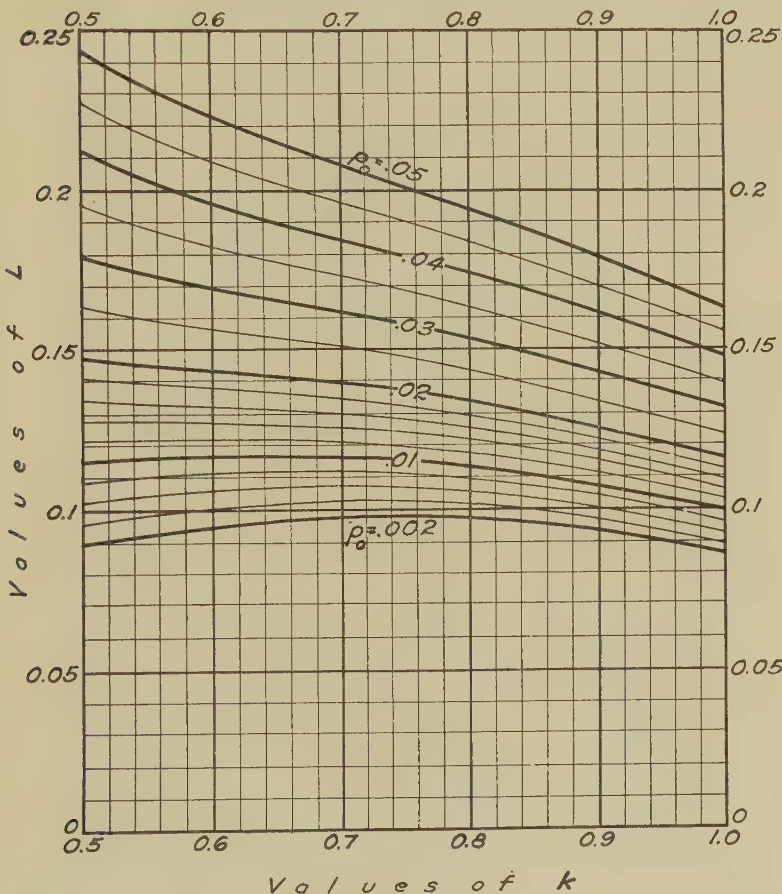
INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 56 (or 57, 58 or 59, according to value of d'/t in the member being designed). The value of p_o used in those diagrams must be modified as follows:

p_o used in Diagram 56 must be divided by 0.79 ($d' = 0.05t$).
p_o used in Diagram 57 used without modification ($d' = 0.1t$).
p_o used in Diagram 58 must be divided by 1.306 ($d' = 0.15t$).
p_o used in Diagram 59 must be divided by 1.78 ($d' = 0.2t$).

(Instructions continued under Part Two.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 60.—(PART TWO)—CASE II—3000-LB. CONCRETE—n = 10

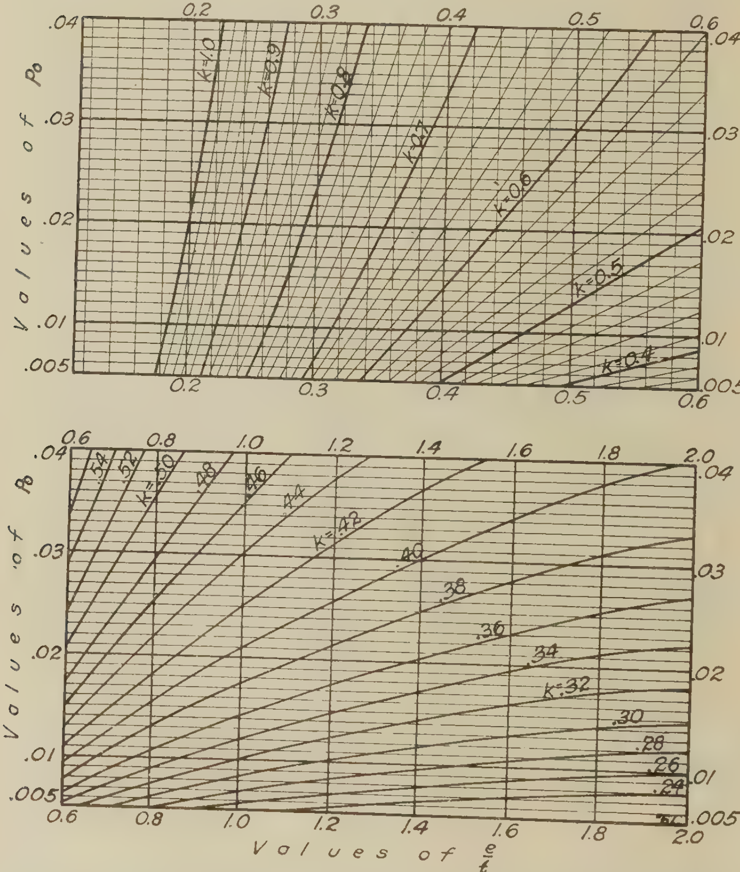


(Instructions continued from preceding page.)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_o. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

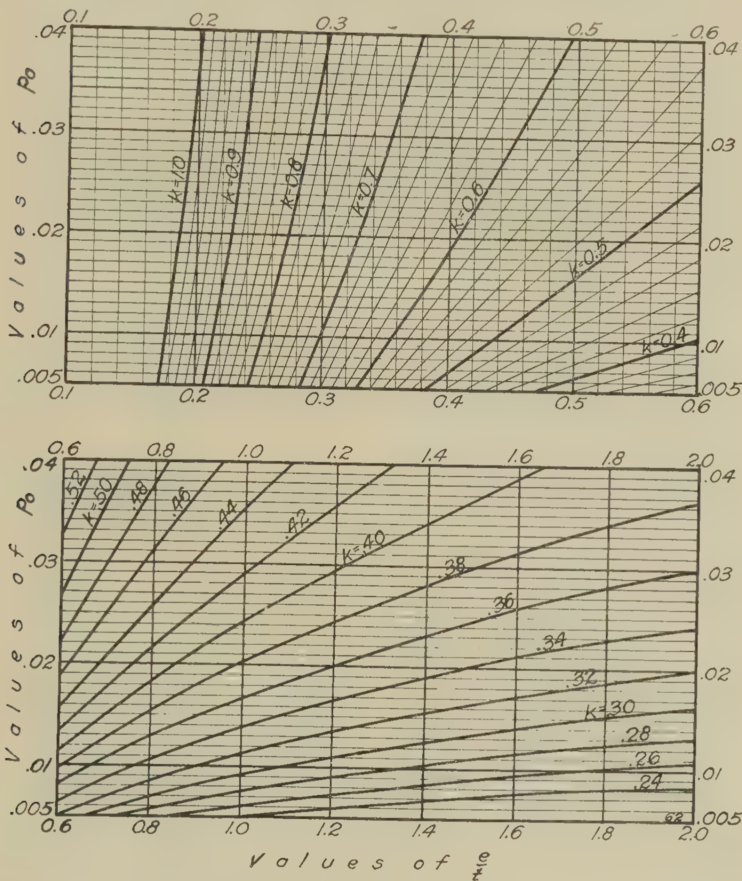
DIAGRAM 61.—CASE II—3750-LB. CONCRETE $n = 8$—$d' = 0.05t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

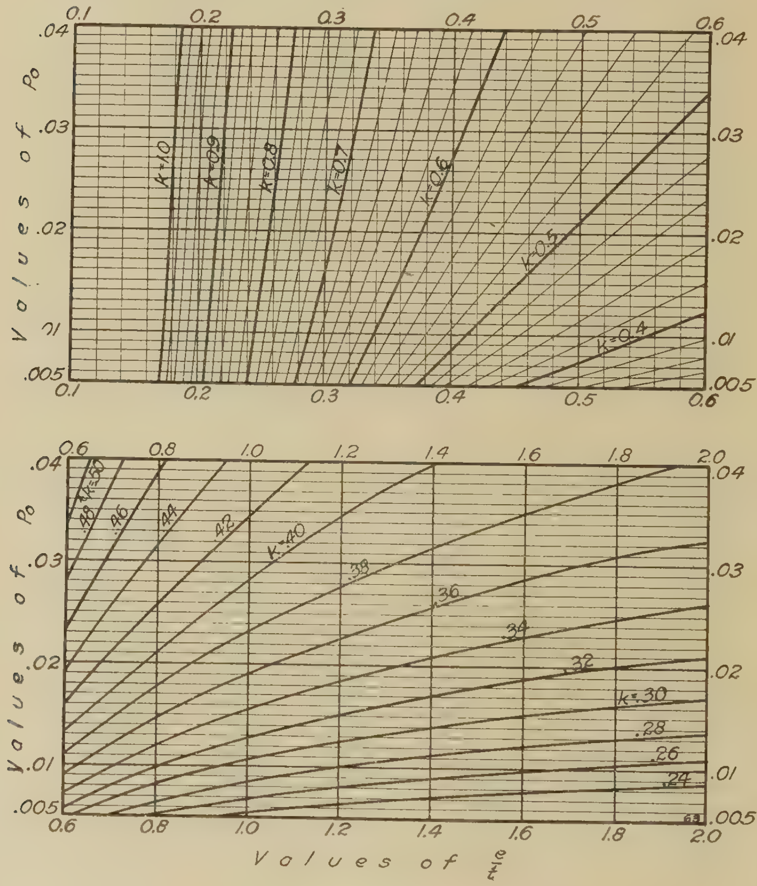
DIAGRAM 62.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.1t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

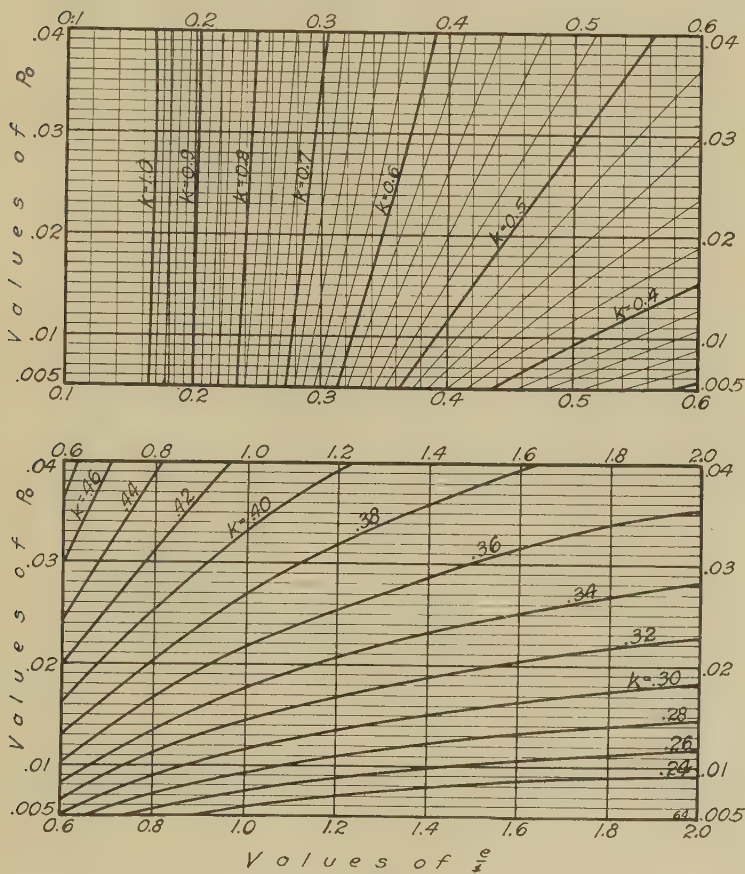
DIAGRAM 63.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.15t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_o on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

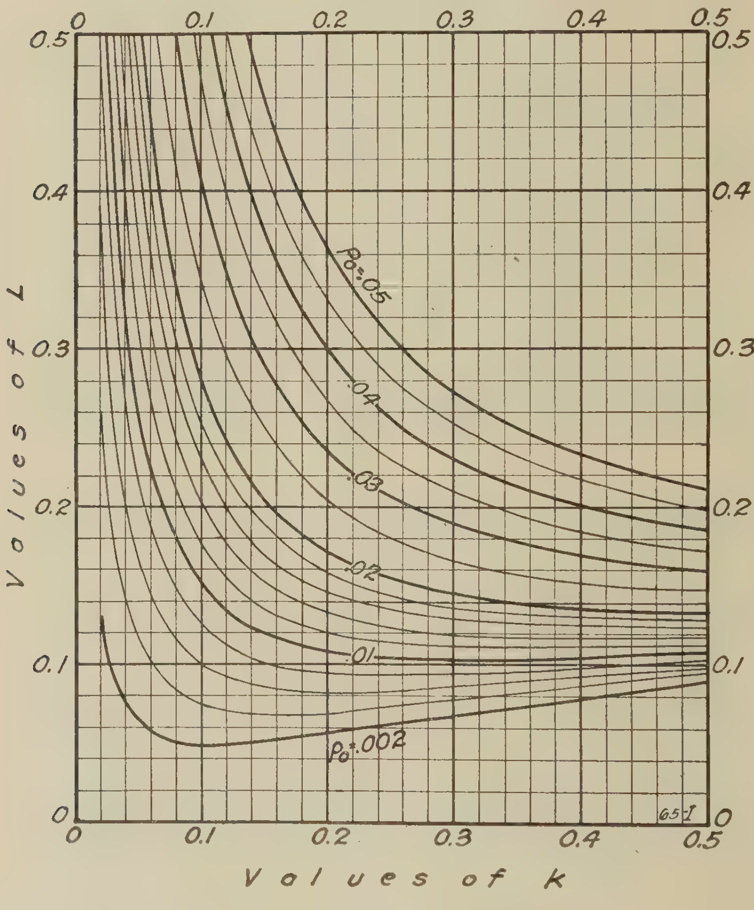
DIAGRAM 64.—CASE II—3750-LB. CONCRETE—$n = 8$—$d' = 0.2t$



INSTRUCTIONS FOR USE.—Enter the diagram with the value of e/t and proceed vertically to an intersection with a horizontal line drawn through an assumed value of p_0 on the left or right marginal scale. Read off on the inclined scales the values of k. With this value of k enter Diagram 65. (See also general note under Diagram 20.)

BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS

DIAGRAM 65.—(PART ONE)—CASE II—3750-LB. CONCRETE—$n = 8$

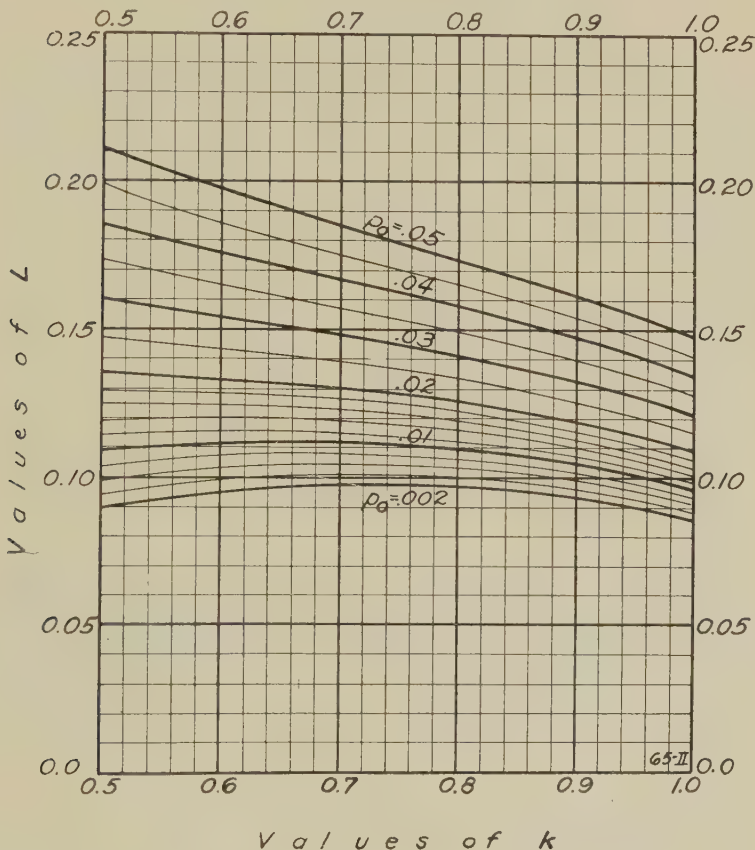


INSTRUCTIONS FOR USE.—The value of k must first be obtained from Diagrams 61 (or 62, 63 or 64, according to value of d'/t in the member being designed). The value of po used in those diagrams must be modified as follows:

po used in Diagram 61 must be divided by 0.79 ($d' = 0.05t$).
po used in Diagram 62 used without modification ($d' = 0.1t$)
po used in Diagram 63 must be divided by 1.306 ($d' = 0.15t$)
po used in Diagram 64 must be divided by 1.78 ($d' = 0.2t$)

(Instructions continued under Part Two.)

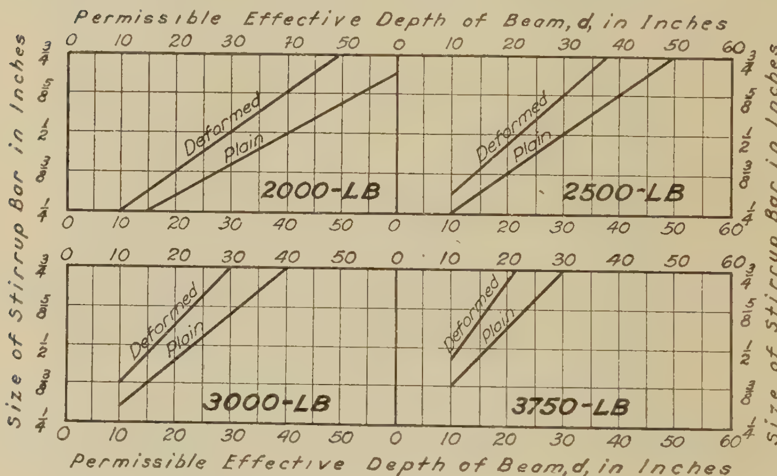
BENDING AND DIRECT COMPRESSION—RECTANGULAR SECTIONS
DIAGRAM 65.—(PART TWO)—CASE II—3750-LB. CONCRETE—n = 8



(Instructions continued from preceding page.)

Enter the diagram with the value of k and proceed vertically to an intersection with the sloping index line corresponding to the modified value of p_o. From this intersection pass horizontally to the left or right marginal scale and read off the value of L. Solve formulas (31) and (32) for the stresses in the concrete and reinforcing steel. (See also general note under Diagram 20.)

DIAGRAM 66.—MAXIMUM SIZE OF STIRRUP BARS



This diagram is based on the requirement that a tensile stress of 16,000 lb. per sq. in. in the stirrup or web reinforcement bar can be developed by bond on the surface area of the bar embedded within the upper or lower half of the beam at a unit stress of 0.04 $f'c$ for plain bars or of 0.05 fc' for deformed bars. The diagram is based on the usual U-shaped stirrups, anchored at one face of the beam by bending around the longitudinal bars and at the other by means of hooked ends. The diagram is based on a length of hook such that if straightened out, the end of the bar would project 5 in. beyond the surface of the beam. In case stirrup bars larger than the values obtained by this diagram are used the length of hook must be increased or the tensile stress reduced to that which can be developed by bond on the surface of the bars, including the hook, embedded within the half depth of beam.

For vertical stirrups the maximum size of stirrup for any effective depth of beam is obtained directly from that rectangle in the diagram corresponding to the strength of concrete to be used. For inclined web members the maximum size for any effective depth will be larger than that given by the diagram and may be computed by the formula—

$$Di = Dv \frac{(d-1) \operatorname{cosec} \alpha + 6}{d + 5}$$

in which Di = maximum size of inclined web member.

Dv = maximum size of vertical stirrup by diagram 66.

d = effective depth of beam.

α = angle between inclined web member and the horizontal.

TABLE 67.—VALUES OF NA_v FOR U-SHAPED STIRRUPS

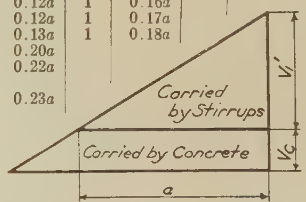
Number of Stirrups at One End	Size of Stirrup					
	¼ in. round	⅜ in. round	½ in. round	½ in. square	⅝ in. round	¾ in. round
20	1.96	4.42	7.85	10.00	12.27	17.67
19	1.87	4.20	7.46	9.50	11.66	16.79
18	1.77	3.97	7.07	9.00	11.04	15.90
17	1.67	3.75	6.67	8.50	10.43	15.02
16	1.57	3.53	6.28	8.00	9.82	14.14
15	1.47	3.31	5.89	7.50	9.20	13.25
14	1.37	3.09	5.50	7.00	8.59	12.37
13	1.28	2.87	5.10	6.50	7.98	11.49
12	1.18	2.65	4.71	6.00	7.36	10.60
11	1.08	2.43	4.32	5.50	6.75	9.72
10	0.98	2.21	3.93	5.00	6.14	8.84
9	0.88	1.99	3.53	4.50	5.52	7.95
8	0.79	1.77	3.14	4.00	4.91	7.07
7	0.69	1.55	2.75	3.50	4.30	6.19
6	0.59	1.32	2.36	3.00	3.68	5.30
5	0.49	1.10	1.96	2.50	3.07	4.42
4	0.39	0.88	1.57	2.00	2.45	3.53
3	0.29	0.66	1.18	1.50	1.84	2.65
2	0.20	0.44	0.79	1.00	1.23	1.77
1	0.10	0.22	0.39	0.50	0.61	0.88

INSTRUCTIONS FOR USE.—The areas given in this table are the right sectional areas of two legs each for the size of stirrup bar shown in the heading and for the number of stirrups given at the left.

The number of stirrups is the number at *each* end for a uniformly loaded beam,or the number of stirrups within the base distance a, in Fig. 7 or 9, of the trapezoid under the shear curve for which stirrups are being designed by formulas (117) or (118).

TABLE 68.—SPACING OF U-SHAPED STIRRUPS—CASES II AND III

Number of Stirrups at One End	Distance, First Stirrup to Face of Support	Spacing, Center to Center of Stirrups, in Terms of a									
		1st Group		2nd Group		3rd Group		4th Group		5th Group	
		No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing
20	0.013a	8	0.03a	7	0.04a	2	0.06a	1	0.08a	1	0.11a
19	0.013a	7	0.03a	6	0.04a	3	0.06a	1	0.08a	1	0.12a
18	0.014a	6	0.03a	5	0.04a	4	0.06a	1	0.08a	1	0.12a
17	0.015a	5	0.03a	5	0.04a	4	0.06a	1	0.09a	1	0.13a
16	0.016a	3	0.03a	5	0.04a	5	0.06a	1	0.09a	1	0.13a
15	0.017a	2	0.03a	5	0.04a	4	0.06a	2	0.08a	1	0.14a
14	0.018a	5	0.04a	4	0.05a	2	0.08a	1	0.09a	1	0.14a
13	0.019a	4	0.04a	3	0.05a	3	0.08a	1	0.09a	1	0.14a
12	0.021a	6	0.05a	3	0.07a	1	0.12a	1	0.15a		
11	0.023a	5	0.05a	3	0.08a	1	0.12a	1	0.15a		
10	0.025a	3	0.05a	4	0.08a	1	0.12a	1	0.16a		
9	0.028a	3	0.06a	3	0.09a	1	0.12a	1	0.17a		
8	0.032a	2	0.07a	3	0.09a	1	0.13a	1	0.18a		
7	0.036a	3	0.08a	2	0.13a	1	0.20a				
6	0.04a	3	0.10a	1	0.15a	1	0.22a				
5	0.05a	2	0.12a	1	0.16a	1	0.23a				
4	0.07a	2	0.16a	1	0.26a						
3	0.09a	1	0.21a	1	0.30a						
2	0.13a	1	0.37a								
1	0.29a										

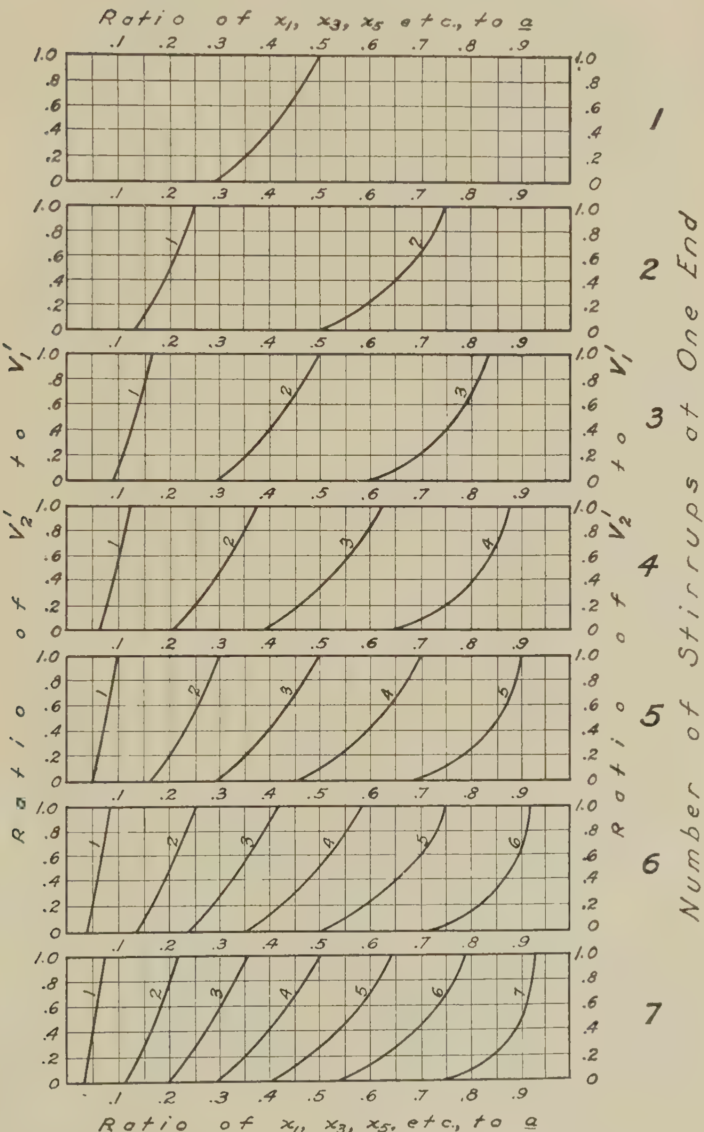


INSTRUCTIONS FOR USE.—Determine from formulas (117) or (118) and Table 67 the value of N, the number of stirrups for one triangle and from formulas (119) or (120) the value of a, the base dimension of the triangle. With these values of N and a enter the table and compute the stirrup spacing.

INSTRUCTIONS FOR USE OF DIAGRAMS 69, 70, 71

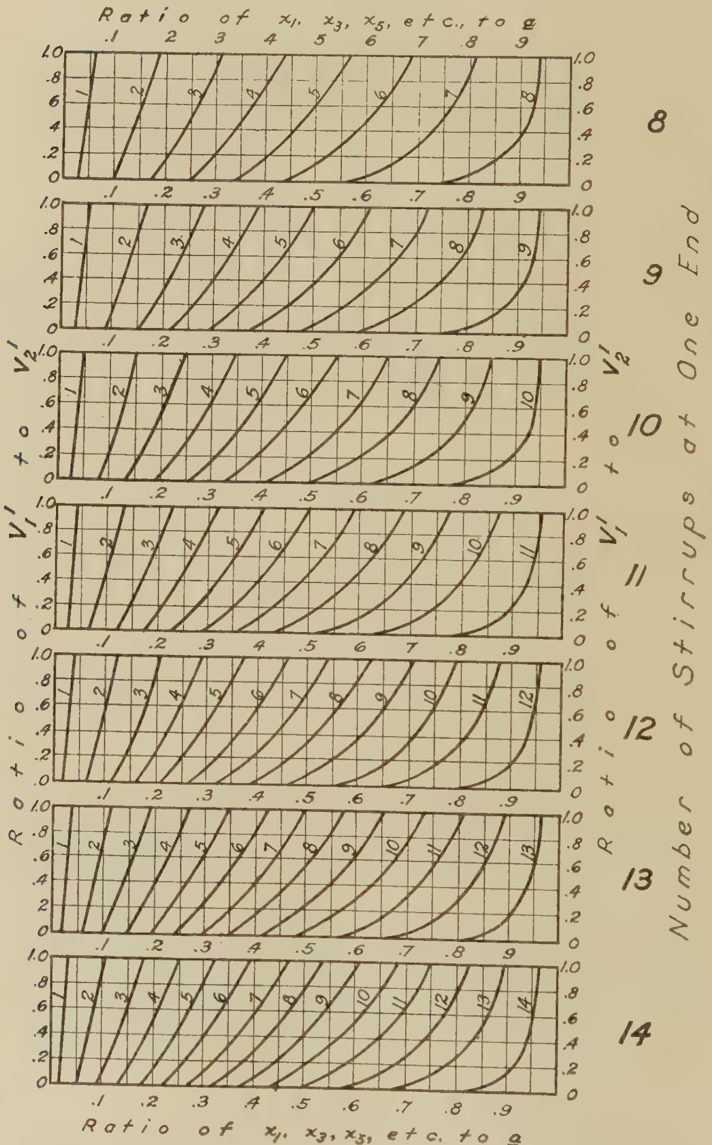
These diagrams give stirrup locations for either vertical or inclined stirrups (at the level of the mid-depth of beam), measured in terms of a from the high end of the trapezoid under the shear curve (see Fig. 7 or 9). Enter the proper rectangle, as determined by N, the number of stirrups computed by equation (117) or (118) and Table 67. Take a straight-edged slip of paper and lay it horizontally across this rectangle at the value of $\frac{V'_2}{V'_1}$ determined from the trapezoid under the shear curve for which stirrups are being designed. Mark on the edge of the slip of paper the intersections with the two marginal lines and with each curve numbered from 1 to N. Now place this slip on Diagram 72 horizontally with the value of a as indicated on the margin equal to a from the shear trapezoid. Read off and record the stirrup locations (as measured from high side of the trapezoid) at each of the marks on the slip. The differences between successive stirrup locations give the spacing between stirrups. Additional stirrups must be provided, if necessary, to reduce the maximum spacing to $\frac{3}{4}d$ for beams in which the unit shearing stresses do not exceed $.06f'c$ or to $\frac{3}{8}d$ for beams with higher shearing stresses. (See 1928 Joint code, Section 803.)

DIAGRAM 69.—SPACING OF 1 TO 7 STIRRUPS



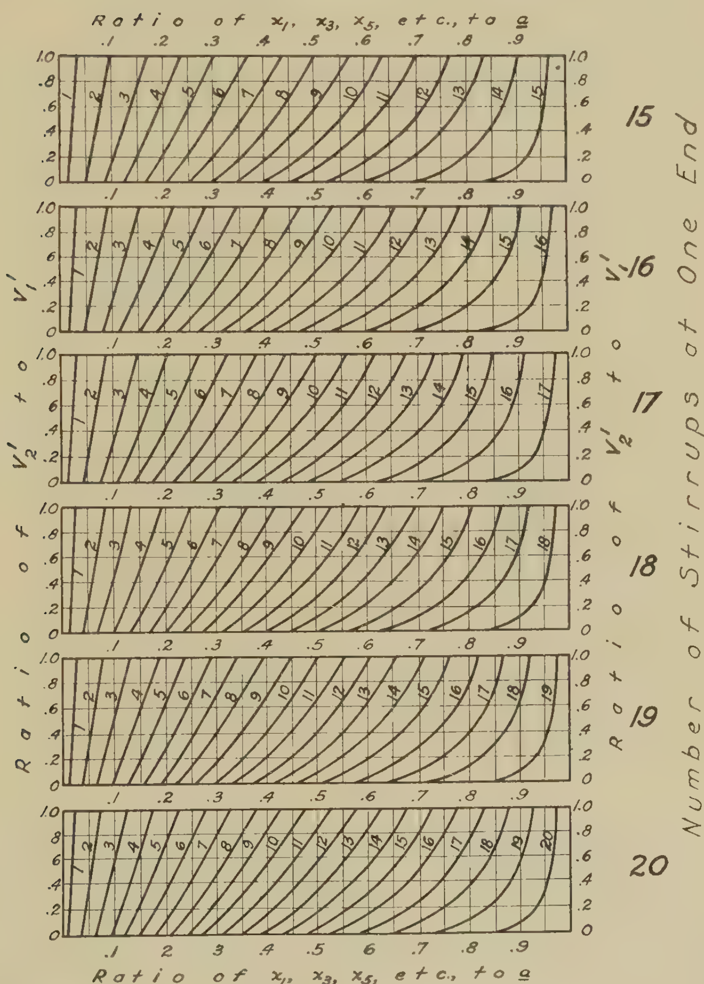
See instructions for use under Table 68.

DIAGRAM 70.—SPACING OF 8 TO 14 STIRRUPS



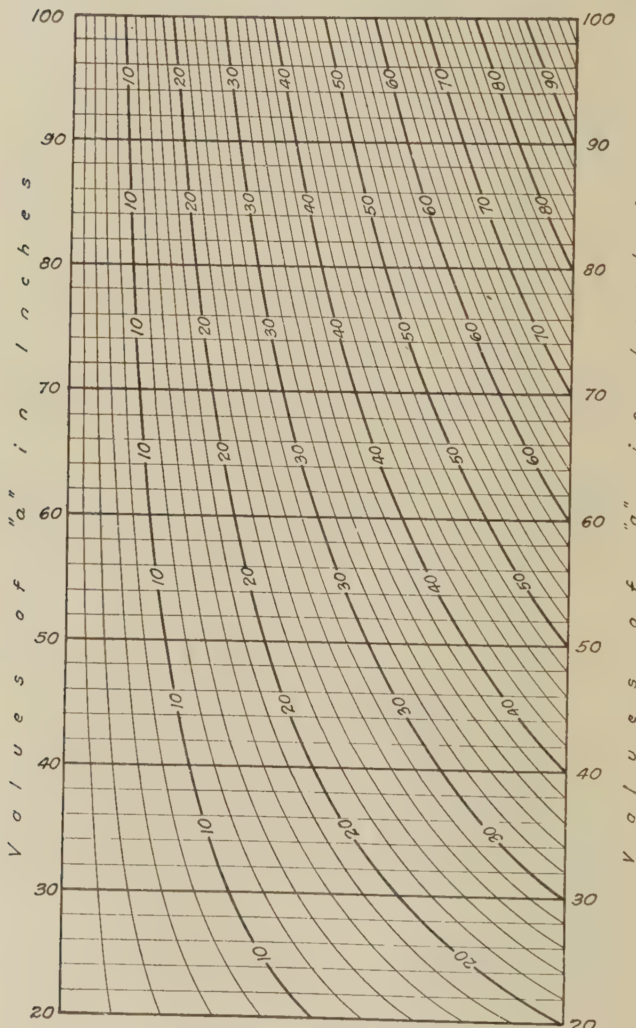
See instructions for use under Table 68.

DIAGRAM 71.—SPACING OF 15 TO 20 STIRRUPS



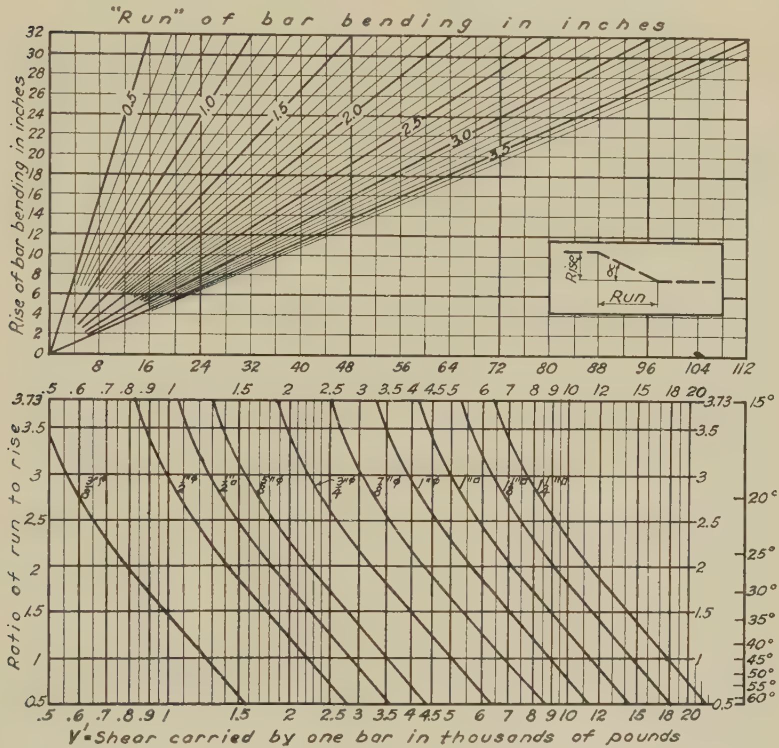
See instructions for use under Table 68.

DIAGRAM 72.—READING CHART FOR DIAGRAMS 69, 70 AND 71



See instructions for use under Table 68.

DIAGRAM 73.—SHEAR VALUES FOR BARS BENT UP IN SINGLE PLANE



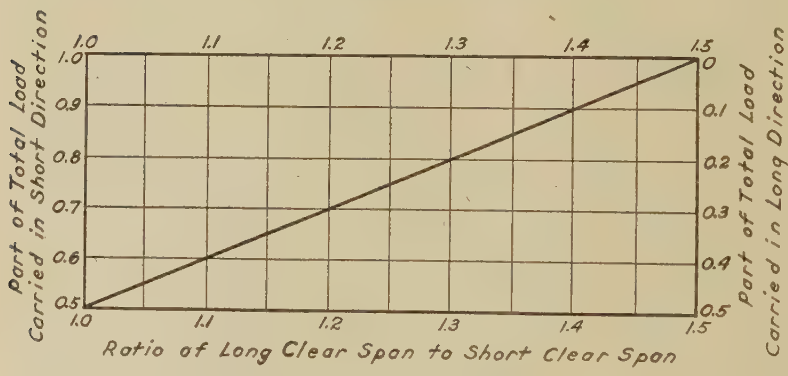
INSTRUCTIONS FOR USE.—This diagram gives designs in accordance with formula (15) in section 803 of the code and excludes any design not in conformity therewith. The upper portion of this diagram is given for convenience in translating "rise" and "run" dimensions into *slope* or ratio of run to rise.

With this ratio enter the lower diagram at the left (or right) margin and proceed horizontally to an intersection with the bar size used. From this intersection drop vertically and read off the shear resistance of one bar on the bottom scale. Multiply this by the number of bars bent up in the single plane to obtain the total shear resistance.

This diagram applies to one or more bars bent up in a single plane at any angle with the horizontal within the limits indicated at the right margin. The indicated shear resistance may be considered effective only in that portion of the beam within which the center ¾ of the bent portion of the bar lies. The total shear resistance of the beam is the sum of resistance of the bent up bars as found above plus the resistance of the concrete by formula (114).

DESIGN OF TWO WAY SLABS SUPPORTED ON BEAMS

DIAGRAM 74.—DISTRIBUTION OF LOAD BETWEEN LONG AND SHORT DIRECTION



INSTRUCTIONS FOR USE.—The same moment coefficients are used for design strips in two-way slabs within the middle half of the slab in each direction as are used for beams under the same conditions of support and restraint. In the outer quarters, the reinforcement parallel to the supporting beam may be reduced to 50 per cent of that in the middle half in the same direction.

For any ratio of length to breadth this diagram gives the proportion of the total dead and live load which the slab must be designed to carry and which it transmits to the supporting beam. The supporting beam must be designed to carry in addition to its own weight and superimposed live load a uniform load throughout its length equal to the load per foot brought to it by the middle strips on either side.

ARRANGEMENT OF REINFORCEMENT IN FLAT SLAB FLOORS.

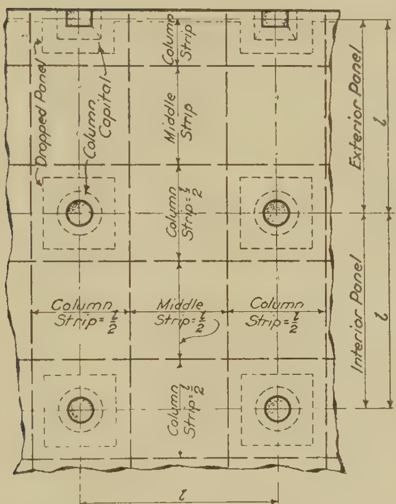


FIG. 14.—TWO-WAY.

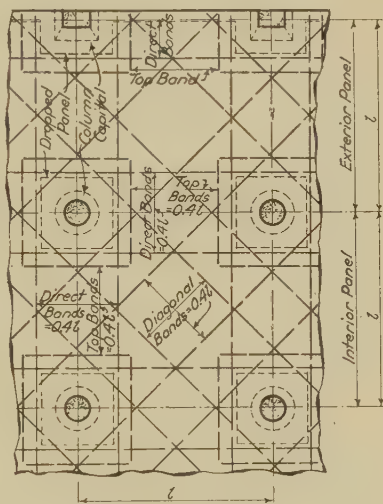


FIG. 15.—FOUR-WAY.

INSTRUCTIONS FOR USE OF DIAGRAMS 77, 79, 81, ETC., TO 91

These diagrams are to be used for square panels, of either two-way or four-way arrangement, surrounded by other interior or exterior panels of the same size, shape and loading. They apply to interior panels when used in conjunction with Table 75 and to exterior panels with full size capitals when used in conjunction with Tables 75 and 76 jointly as described in the notes under Table 76. Diagrams 79, 83, 87 and 91 are based on the use of forms giving column capitals 0.225 l in diameter. Diagrams 77, 81, 85 and 89 are based on the use of metal forms with the usual 6-in. intervals between successive sizes.

In selecting bars to fit the areas determined from these diagrams and Tables 75 and 76 the area actually provided in bent bars must not be reduced as this would result in a deficiency over the column head. It is entirely proper to reduce the actual area in the straight bars of any strip by the amount of the excess in area actually provided in the bent bars of the same strip.

These diagrams are based on a minimum of one inch of fireproofing between all bars and the slab surfaces.

INSTRUCTIONS FOR USE OF DIAGRAMS 78, 80, 82, ETC., TO 92

Enter the diagram at the top with the side dimension of the square panel and drop vertically to an intersection in the upper group of curves with the proper curve for the live load used in design. Read off the volume of concrete by moving horizontally from this intersection to either side scale.

In the same manner drop on the same vertical line to the middle and lower group of curves, successively, and read off the area of formwork and the weight of reinforcing steel on the side scales.

DESIGN COEFFICIENTS FOR FLAT SLAB FLOORS

TABLE 75.—SQUARE INTERIOR PANELS

Diagrams 77, 79, etc., inclusive give slab thickness, column capital diameters, dimensions of square dropped panels and values of A_s for both two-way and four-way flat slabs, with live loads varying from 100 to 300 lb. per sq. ft. and with panel sides from 16 to 24 feet. To complete the design proceed as follows:

The *thickness* of the dropped panel is always equal to one-half of the slab thickness from the diagram.

For *two-way flat slabs* the area of bars in each strip bears the following relation to A_s from the diagrams:

The area of the bent bars in each middle strip is 0.89 A_s.

The area of the straight bars in each middle strip is A_s.

The area of the bent bars in each column strip is 1.56 A_s.

The area of the straight bars in each column strip is 0.78 A_s.

The area of the straight bars, which must be added in the top of the slab over the column head in each column strip is 0.36 A_s.

For *four-way flat slabs* the area of bars in each band, taken at right angles to the direction of the band, bears the following relation to A_s from the diagrams:

The area of bars in each top band, across the direct band, is A_s.

The area of the bent bars in each diagonal band is 0.67 A_s.

The area of the straight bars in each diagonal band is A_s.

The area of the bent bars in each direct band is A_s.

The area of the straight bars in each direct band is 1.22 A_s.

The shearing and compressive unit stresses and the amount of reinforcement at the column head will not require computation where these diagrams are used in the manner set forth above.

The coefficients apply to interior panels surrounded by other interior or exterior panels approximately the same size and shape and subjected to the same loading.

Design Coefficients for Flat Slab Floors

Table 76.—Special Requirements for Square Exterior Panels

For *two-way flat slabs* the area of bars in each strip bears the following relation to A_s from the diagrams:

The area of bent bars for each middle strip perpendicular to the wall is 1.11 A_s.

The area of straight bars for the same strip is 1.12 A_s.

The area of bent bars for each column strip perpendicular to the wall is 1.95 A_s.

The area of the straight bars for the same strip is 0.98 A_s.

The area of the straight bars, which must be added in the top of the slab over the exterior column head is 1.18 A_s.

No additional straight bars in the column strip perpendicular to the wall will need to be added over the interior column head adjacent to the exterior panel.

For *four-way flat slabs* the area of bars in each band bears the following relation to A_s from the diagrams:

The area of bars in the top band across the direct band at the wall side of the panel will be 0.625 A_s.

The area of the bent bars in each diagonal band will be 1.13 A_s.

The area of the straight bars in each diagonal band will be 0.75 A_s.

The area of the straight bars which must be added in the top over the exterior column head in each diagonal band will be 0.22 A_s.

The area of the bent bars in each direct band perpendicular to the wall is 1.67 A_s.

The area of the straight bars in each direct band perpendicular to the wall is 1.11 A_s.

Note.—The slab and dropped panel thicknesses are taken from Diagrams 77, 79, etc., and are the same as for interior panels of the same size, shape and loading.

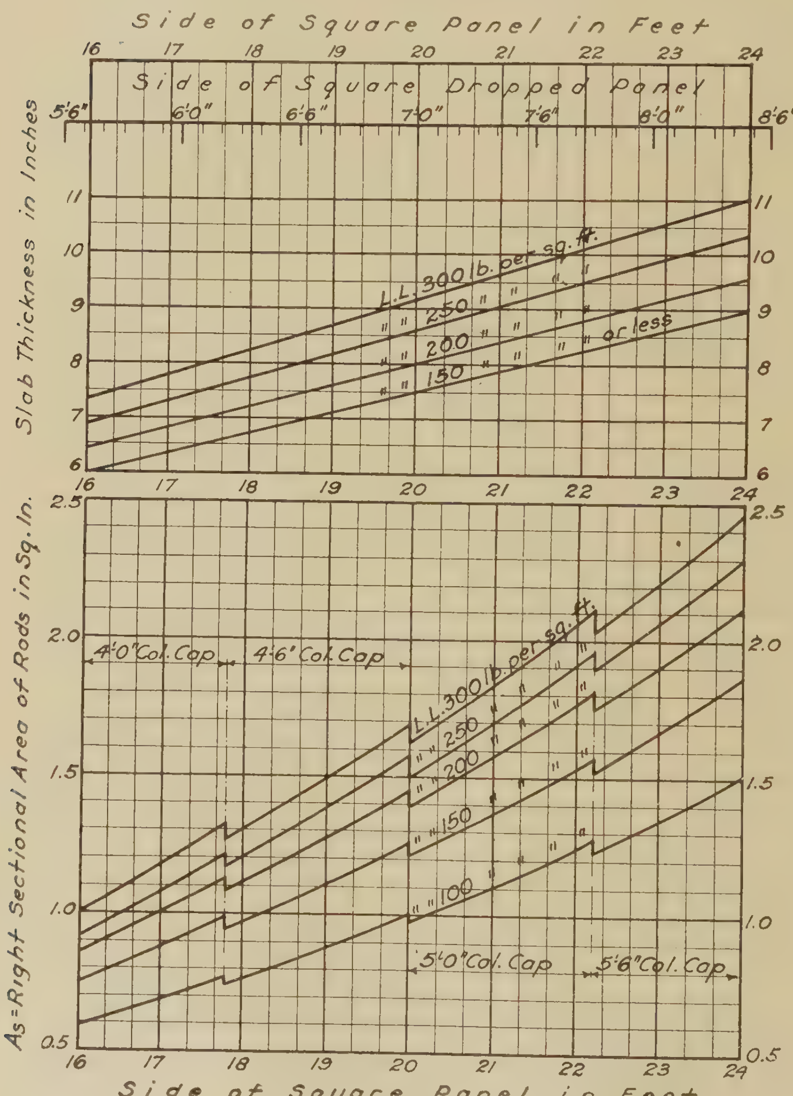
These coefficients apply only to exterior panels surrounded on three sides by other exterior or interior panels of the same size and shape and subject to the same loading. The column capitals at the wall must be the same size as for interior panels, except as cut off by the building line, in order for this table and the accompanying diagrams 77, 79, etc., to apply.

The steel area in strips parallel to the wall will be identical to the steel area found by Table 75 for the corresponding strips of an interior panel, except that the strip lying along the wall will have only a portion of the width of an interior strip and will be designed in accordance with section 1011 of the 1928 Joint code.

Steel areas in the strips in *both* directions in corner panels will be determined solely by this table.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

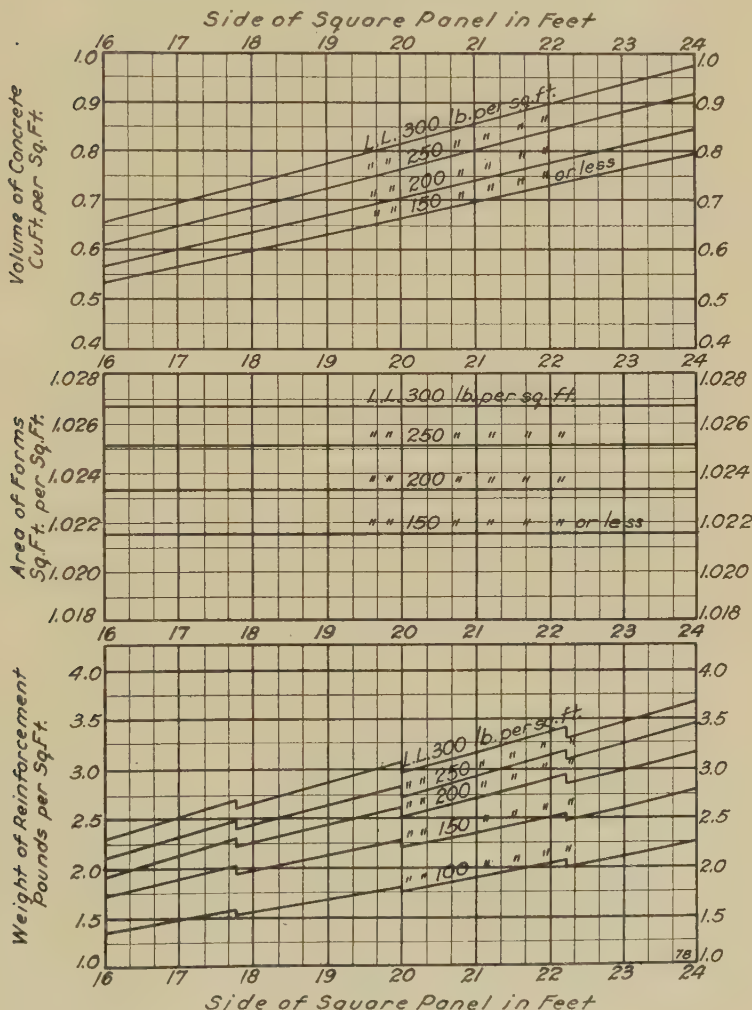
DIAGRAM 77.—2000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 651.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

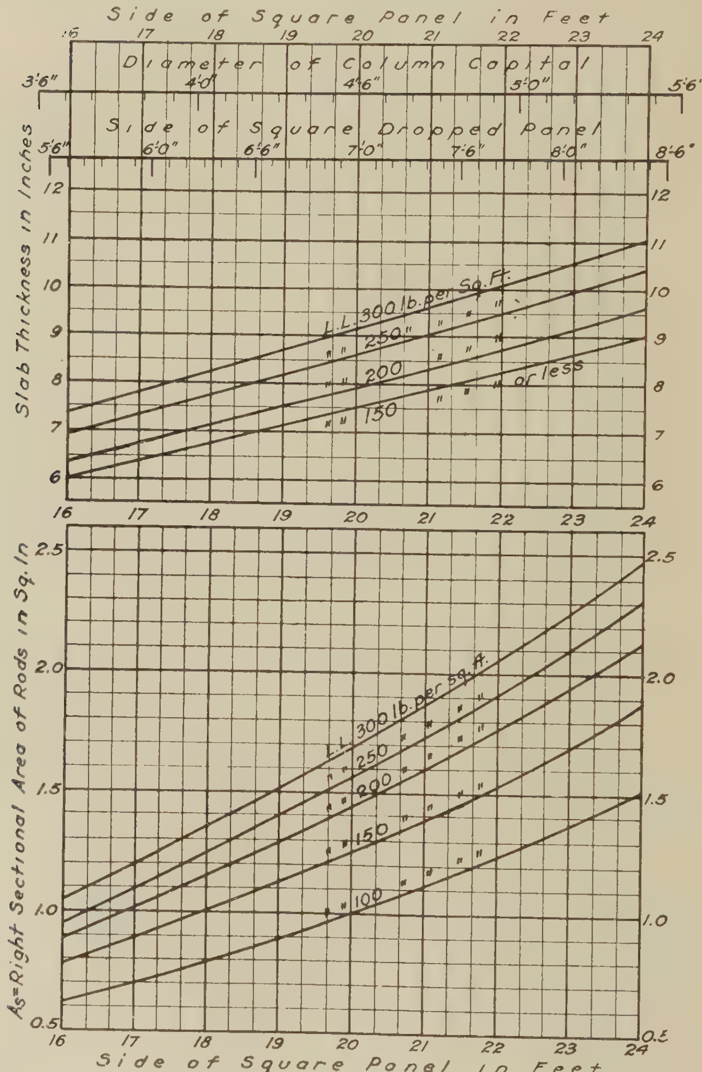
DIAGRAM 78.—2000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 651.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

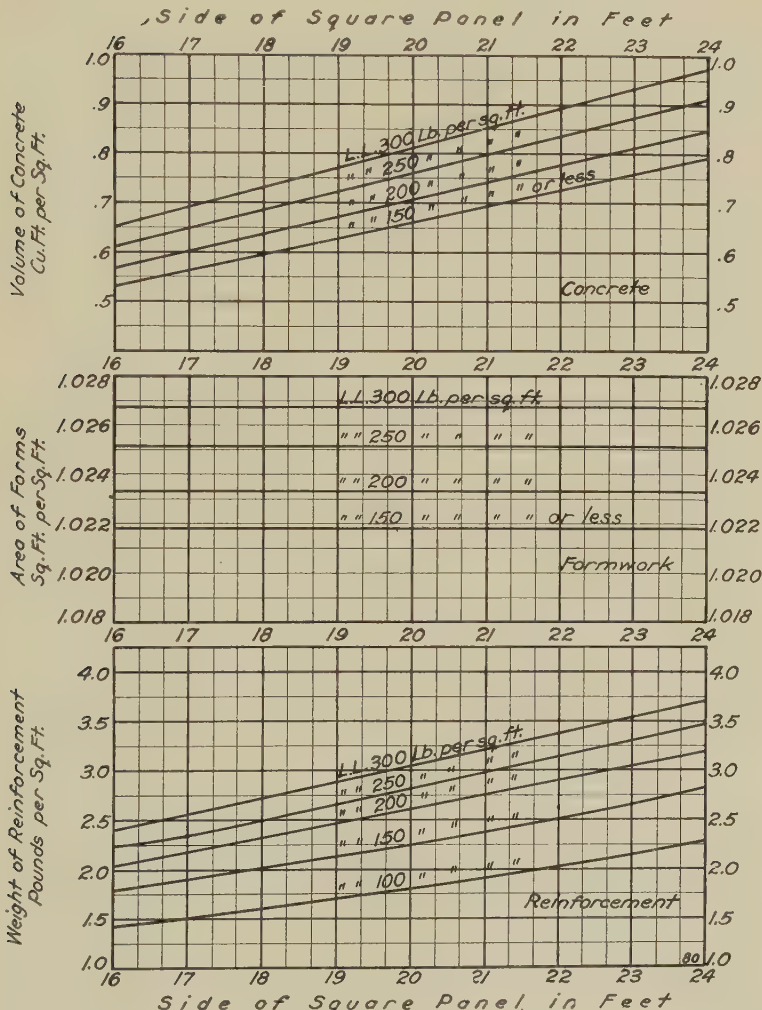
DIAGRAM 79.—2000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 651.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

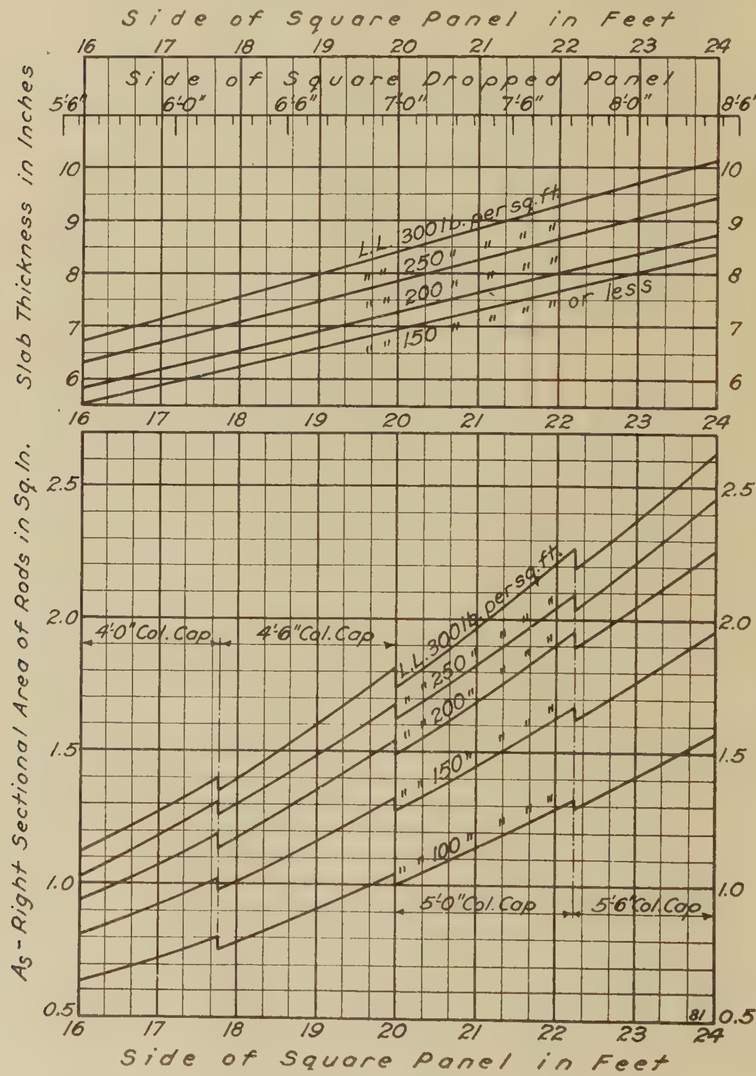
DIAGRAM 80.—2000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 651.

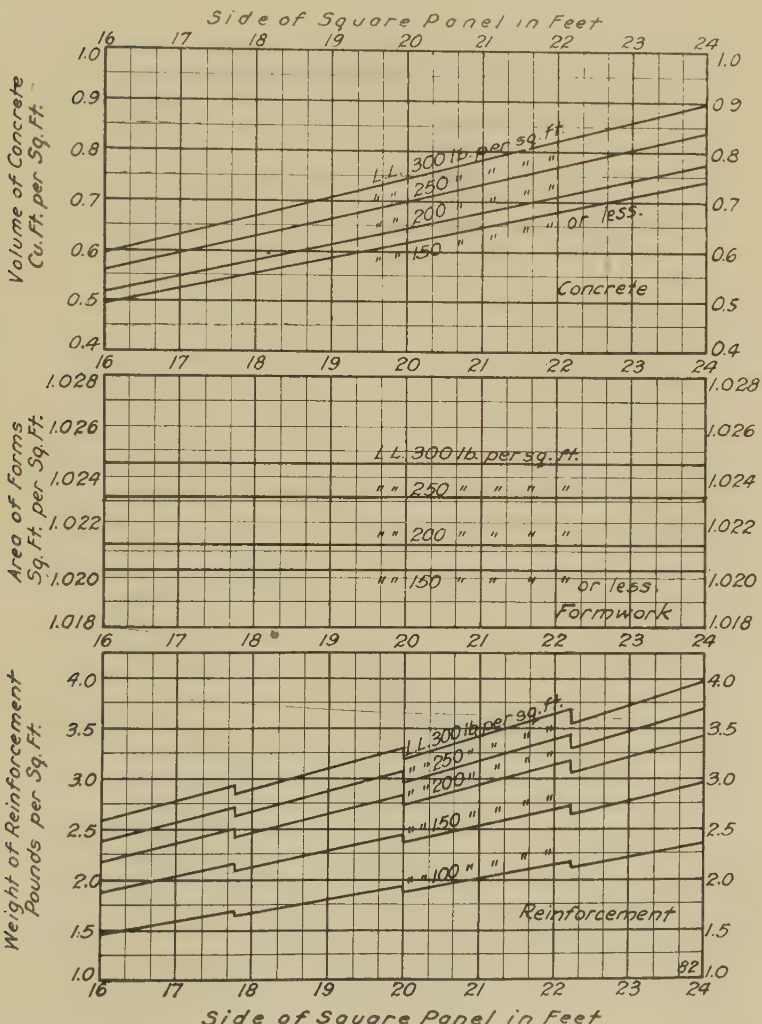
DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 81.—2500-LB. CONCRETE—METAL COLUMN FORMS



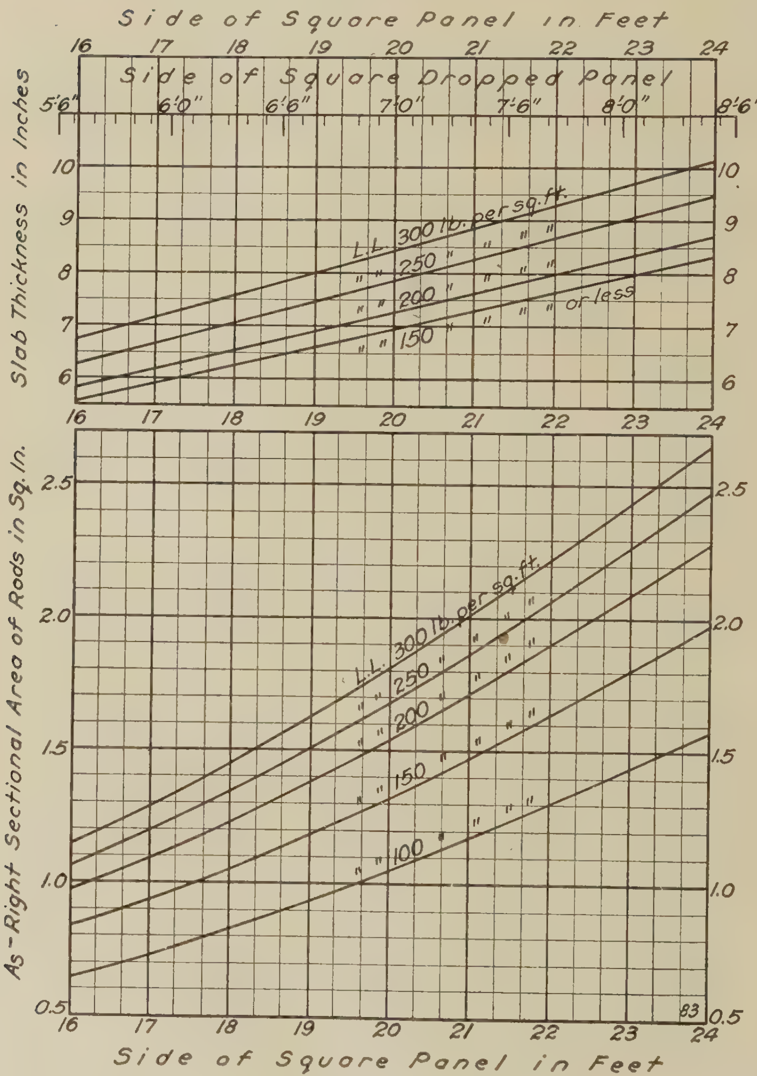
For instructions for use see general note under Fig. 14, page 651.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 82.—2500-LB. CONCRETE—METAL COLUMN FORMS



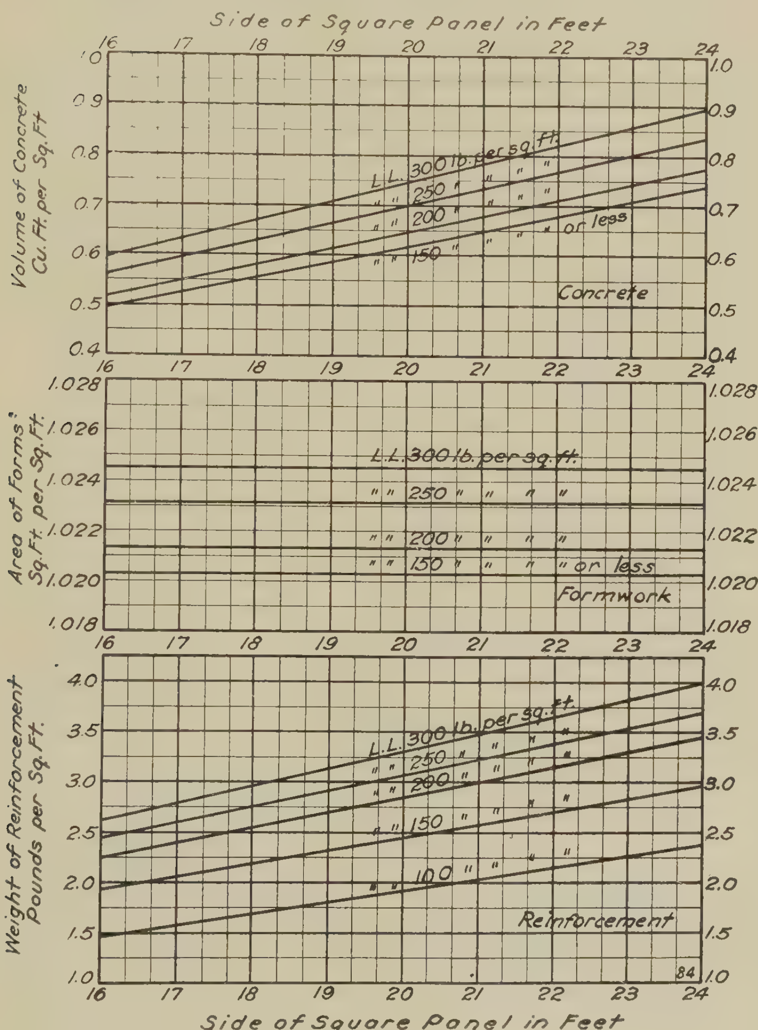
For instructions for use see general note under Fig. 14, page 651.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 83.—2500-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 651.

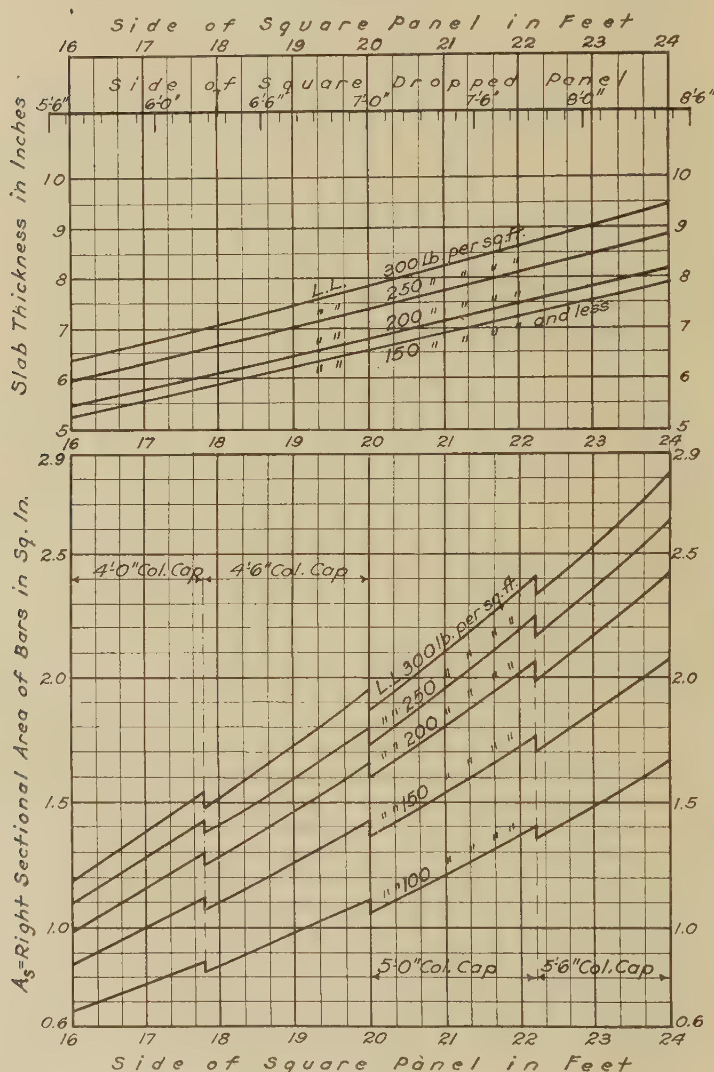
QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 84.—2500-LB. CONCRETE—0.225*l* CAPITAL



For instructions for use see general note under Fig. 14, page 651.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

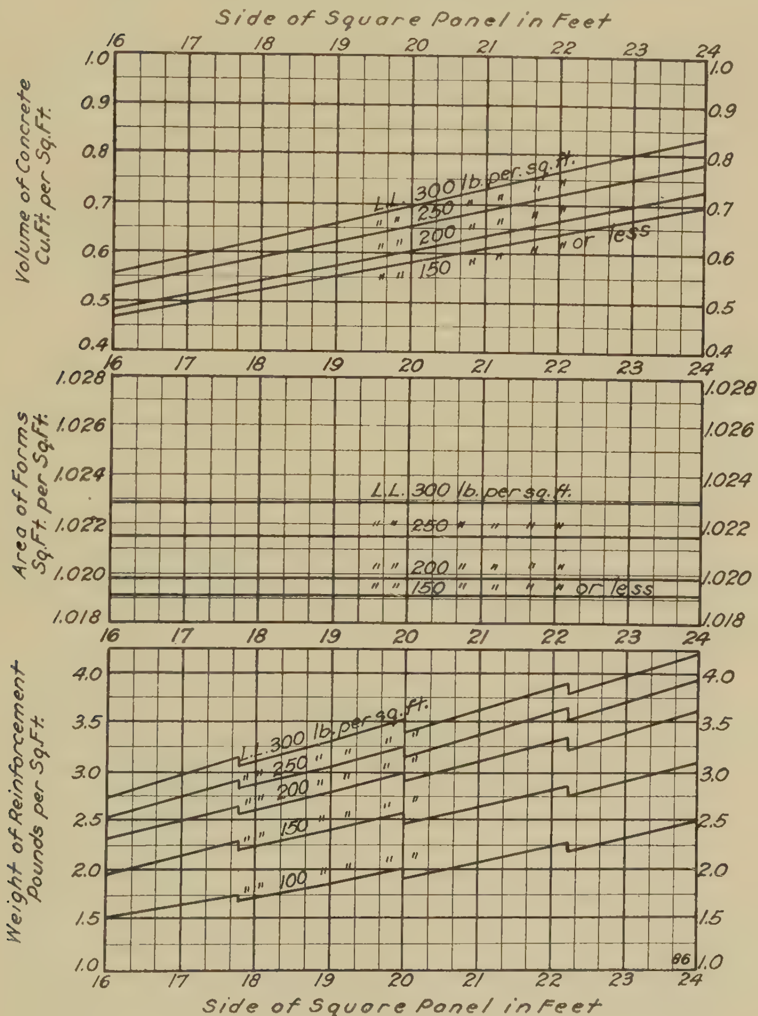
DIAGRAM 85.—3000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 651.

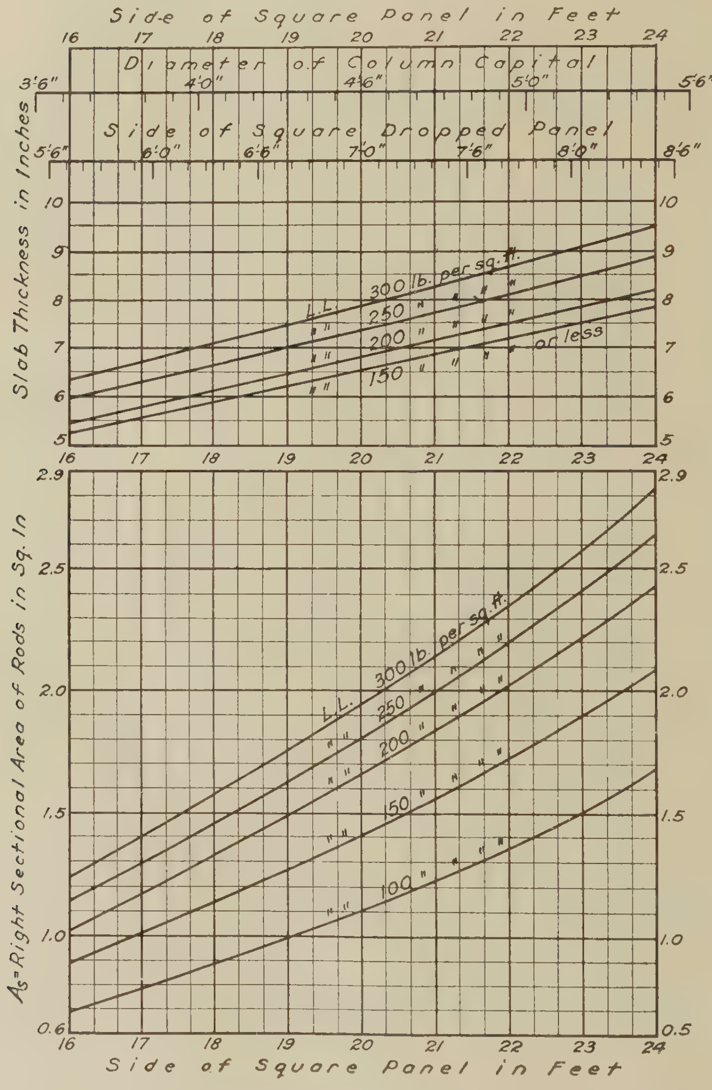
QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 86.—3000-LB. CONCRETE—METAL COLUMN FORMS



For instructions for use see general note under Fig. 14, page 651.

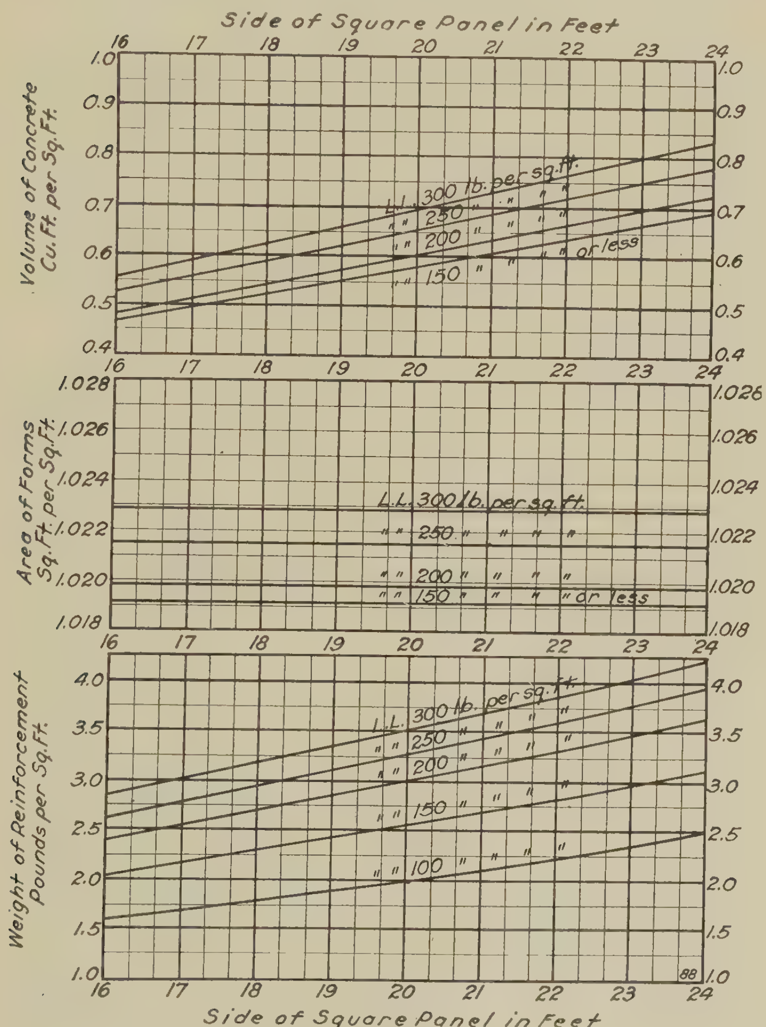
DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 87.—3000-LB. CONCRETE—0.225l CAPITAL



For instructions for use see general note under Fig. 14, page 651.

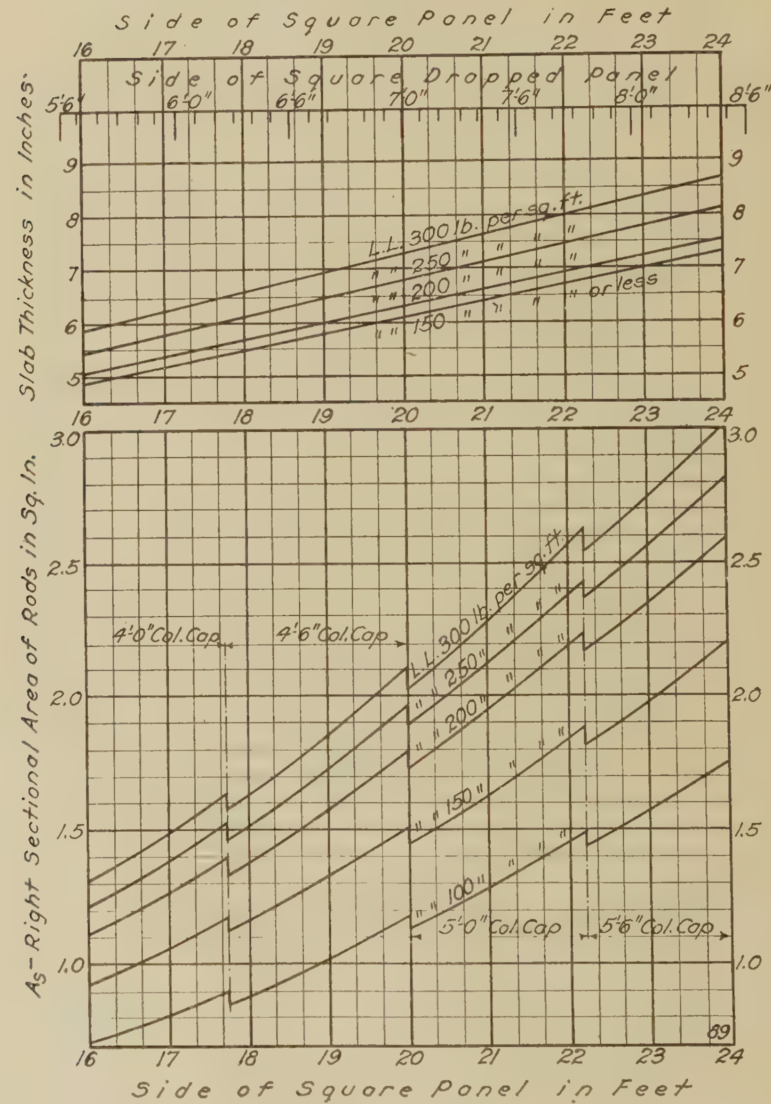
QUANTITIES FOR TWO-WAY AND FOUR-WAY FLAT SLAB FLOORS

DIAGRAM 88.—3000-LB. CONCRETE—0.225l CAPITAL



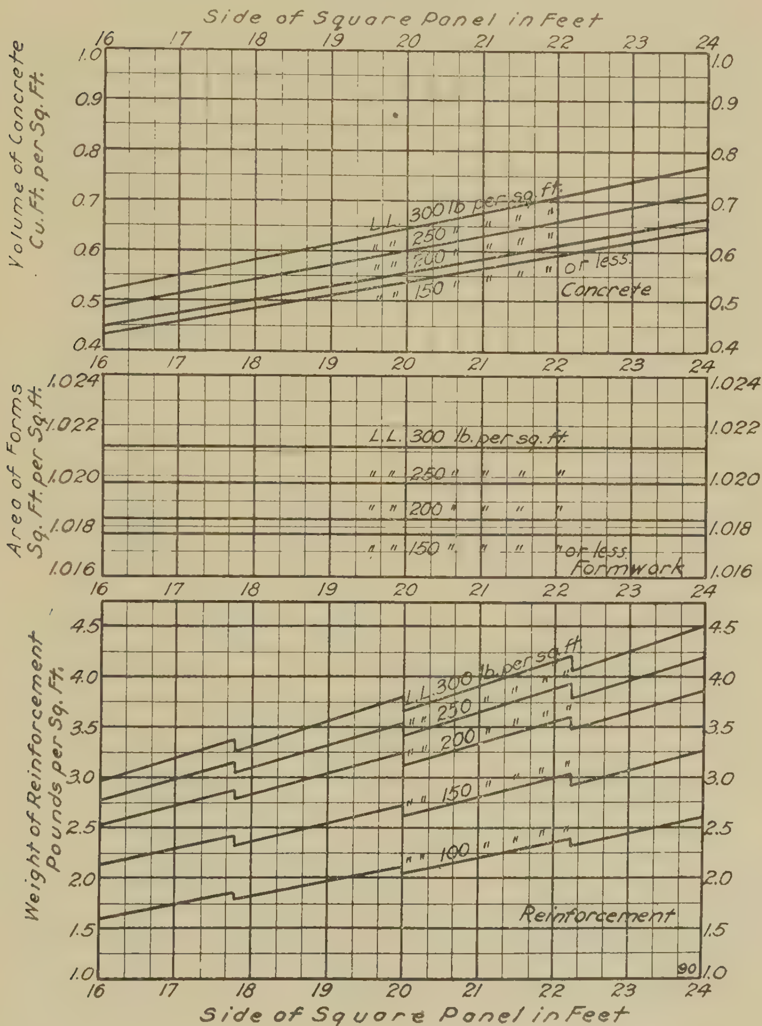
For instructions for use see general note under Fig. 14, page 651.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 89.—3750-LB. CONCRETE—METAL COLUMN FORMS



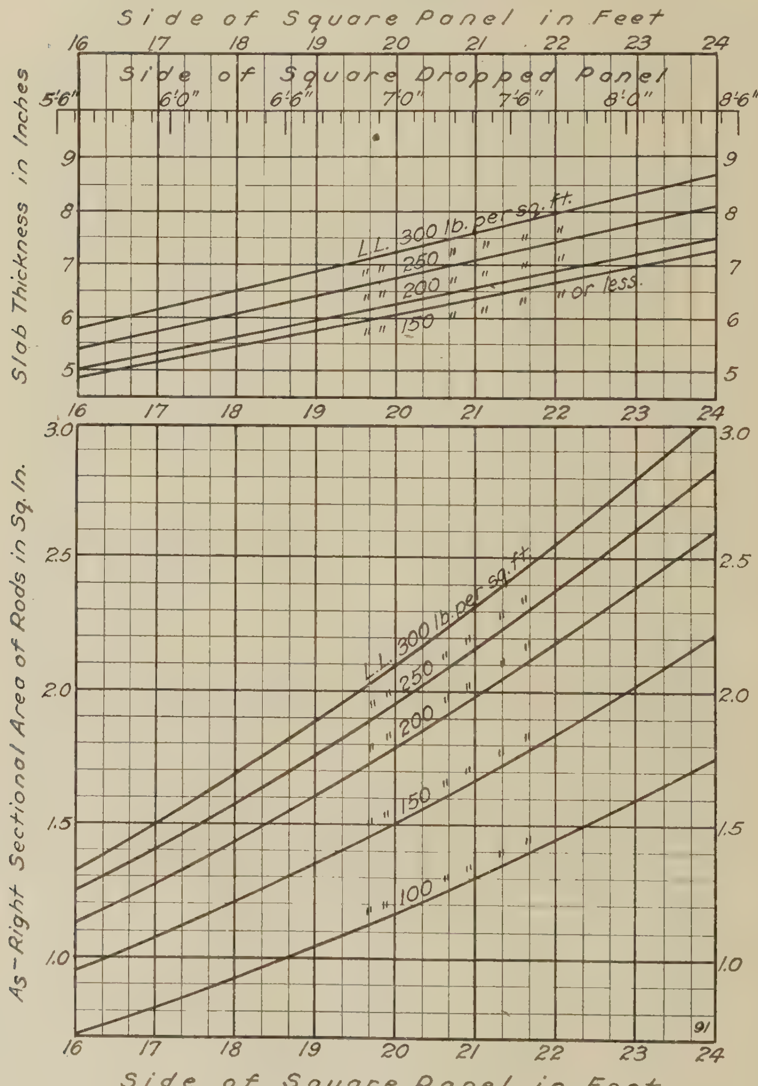
For instructions for use see general note under Fig. 14, page 651.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 90.—3750-LB. CONCRETE—METAL COLUMN FORMS



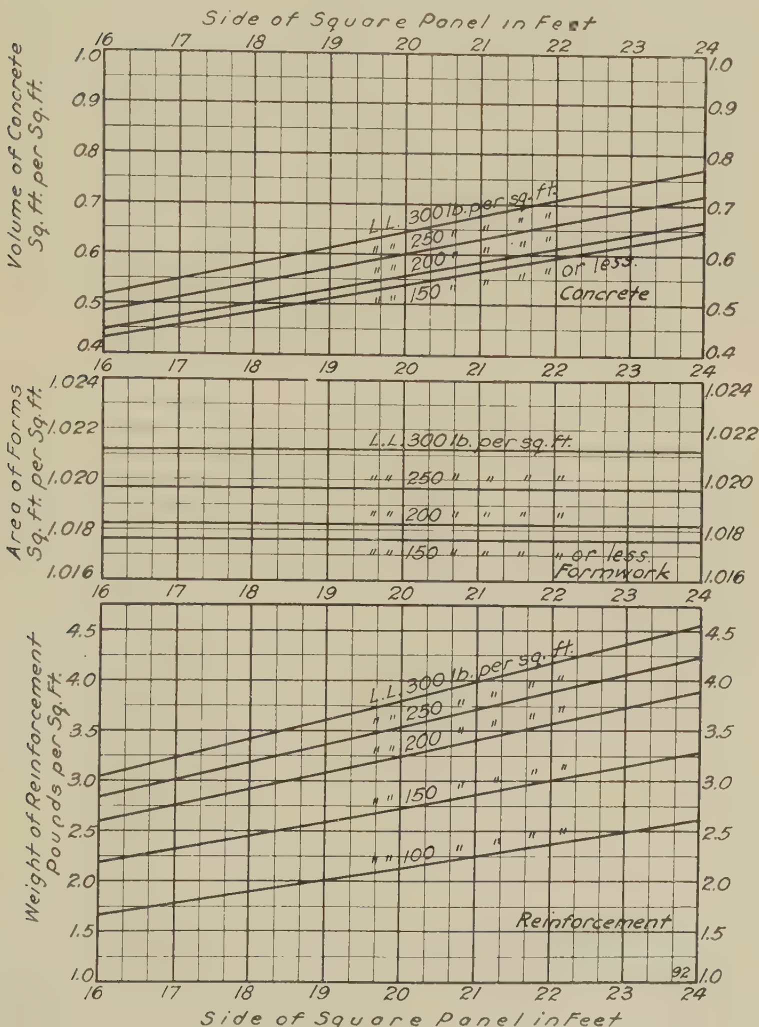
For instructions for use see general note under Fig. 14, page 651.

DESIGN OF TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 91.—3750-LB. CONCRETE—0.225l CAPITAL



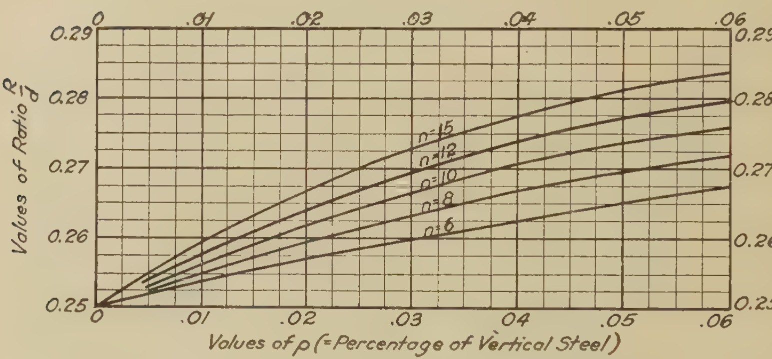
For instructions for use see general note under Fig. 14, page 651.

QUANTITIES FOR TWO-WAY OR FOUR-WAY FLAT SLAB FLOORS
DIAGRAM 92.—3750-LB. CONCRETE 0.225*l* CAPITAL

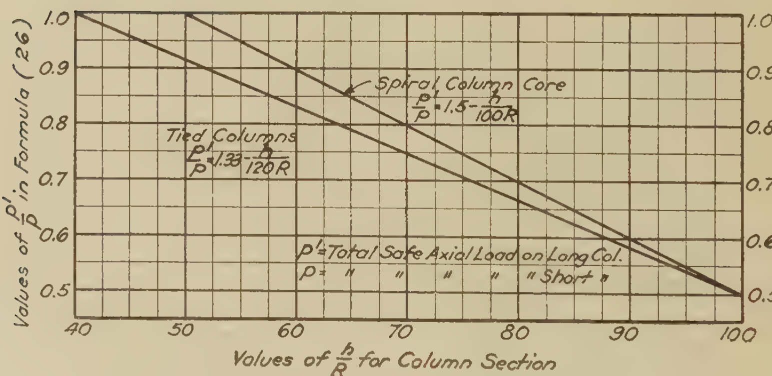


For instructions for use see general note under Fig. 14, page 651.

DIAGRAM 93.—LOAD REDUCTION IN LONG COLUMNS



RADIUS OF GYRATION IN TERMS OF CORE DIAMETER



LOAD FACTOR FOR LONG COLUMNS

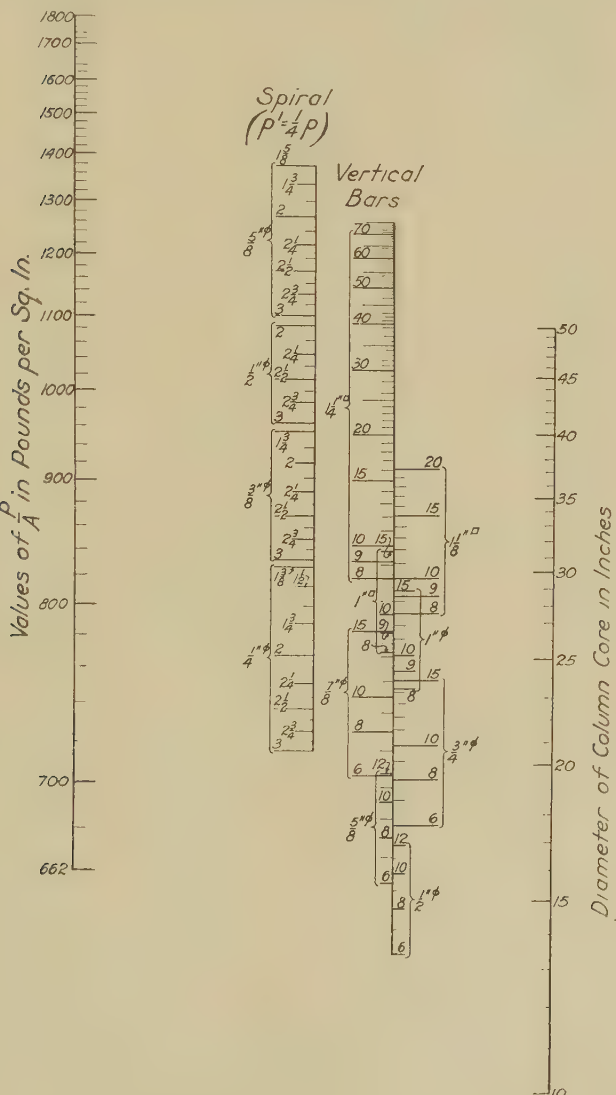
INSTRUCTIONS FOR USE.—This diagram is based on a reduction in load for columns in which the unsupported length exceeds eleven times the least dimension of the column section.

The upper portion gives values of R, the radius of gyration of the transformed section, in terms of the core diameter of a spiral column as used in design. For a tied column the value of R may be taken without considerable waste as that of the concrete alone or as 0.29 times the least dimension of the rectangular column.

The lower portion of the diagram gives the ratio of the load permitted on a long column to that permitted on an ordinary column by the formulas appearing in the diagram. Enter the diagram with the value of h, the unsupported column length divided by R the radius of gyration, move vertically to the proper sloping index line and then horizontally to the marginal scale and read off the allowable proportion of load.

Composite columns are governed by the same load-reduction formula as spiral columns.

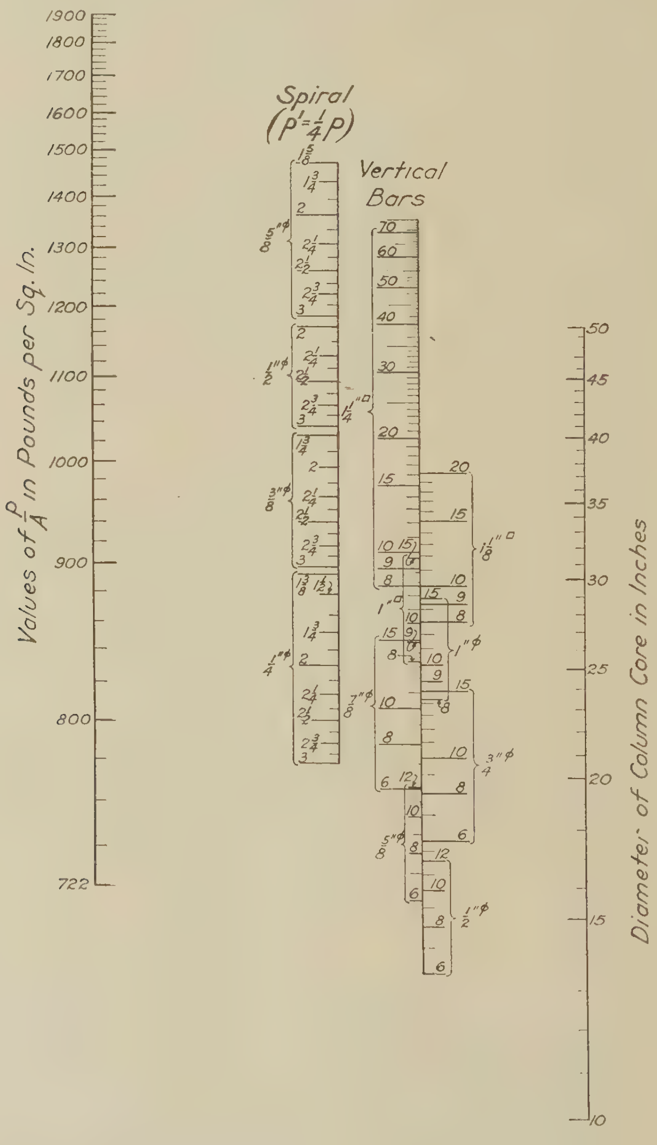
DIAGRAM 94.—DESIGN OF SPIRAL COLUMN—2000-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1280$. (See Section 1103–b of the code.)

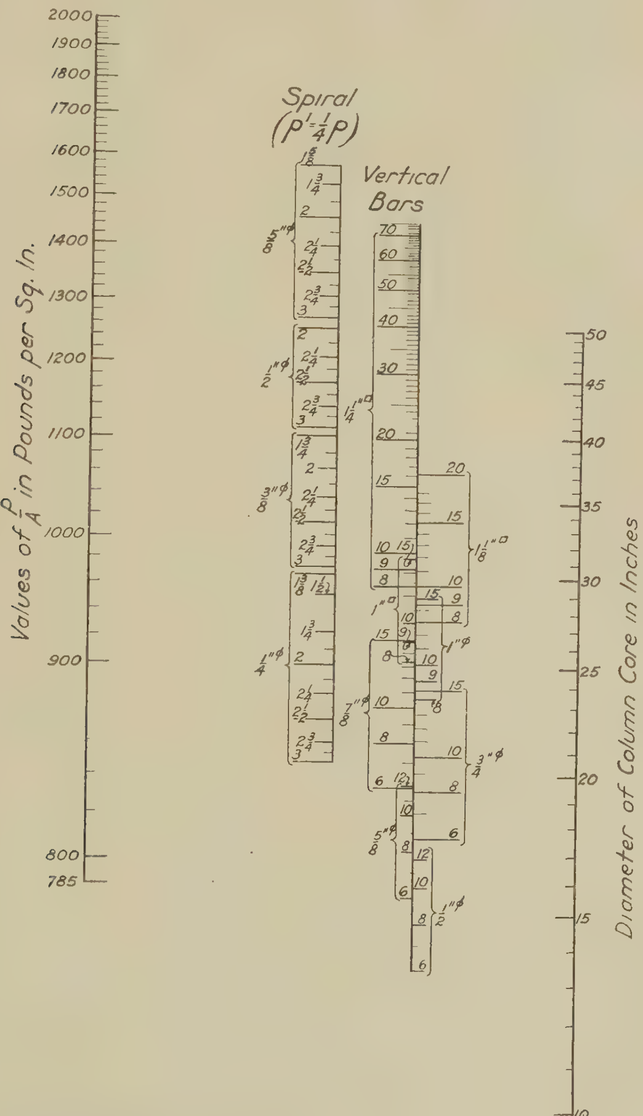
DIAGRAM 95.—DESIGN OF SPIRAL COLUMNS—2500-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1370$. (See Section 1103-b of the code.)

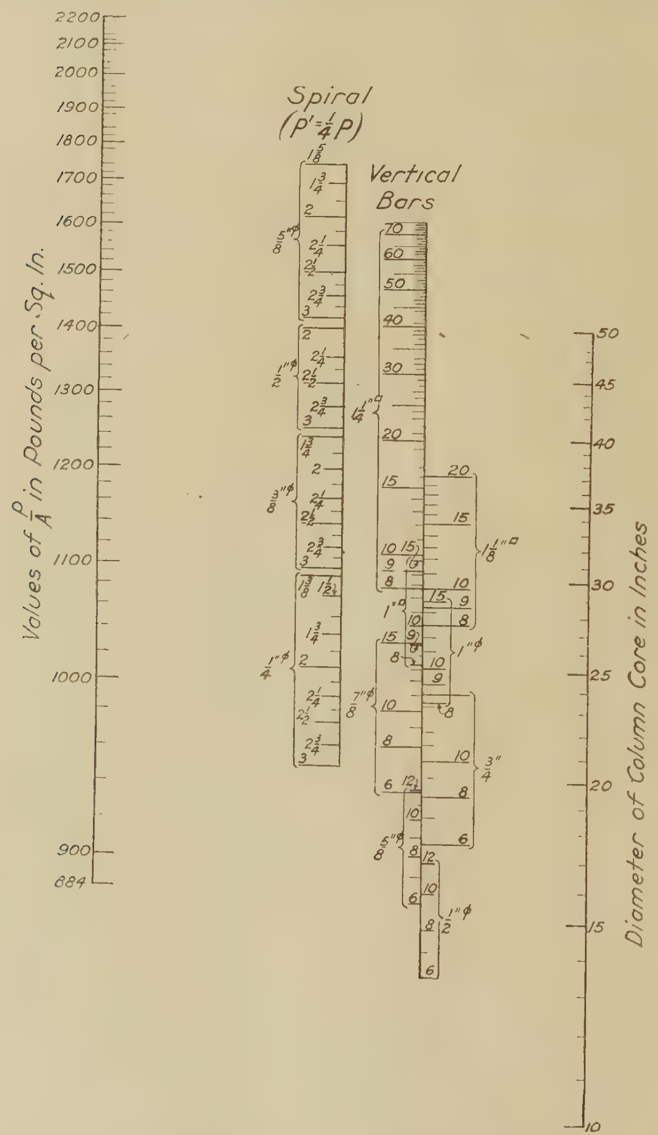
DIAGRAM 96.—DESIGN OF SPIRAL COLUMNS—3000-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1470$. (See Section 1103-b of the code.)

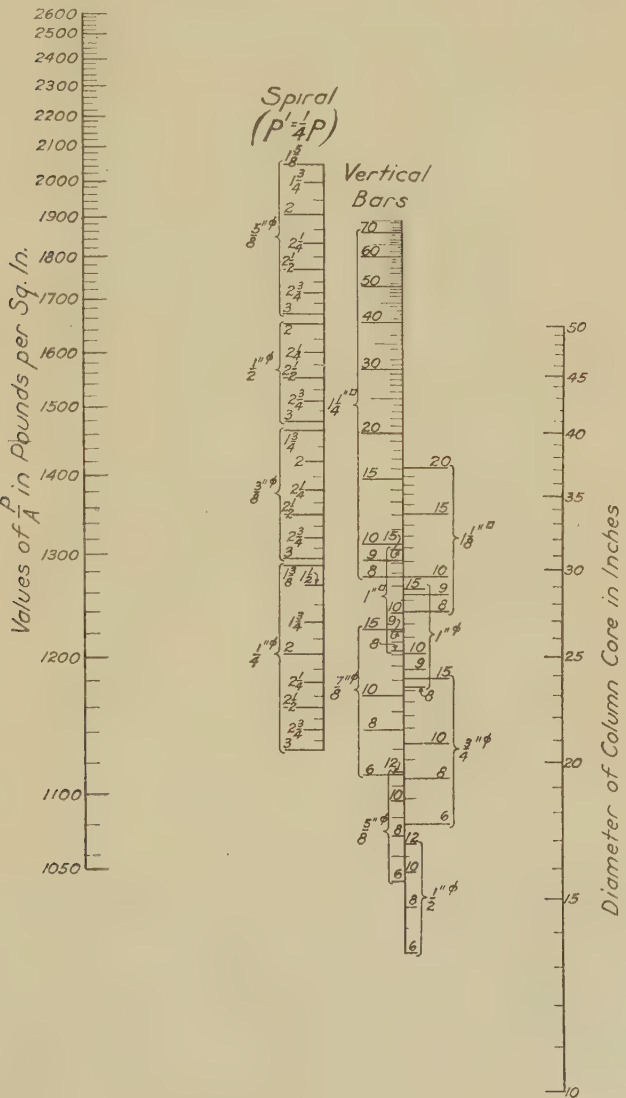
DIAGRAM 97.—DESIGN OF SPIRAL COLUMNS—3750-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1630$. (See Section 1103-b of the code.)

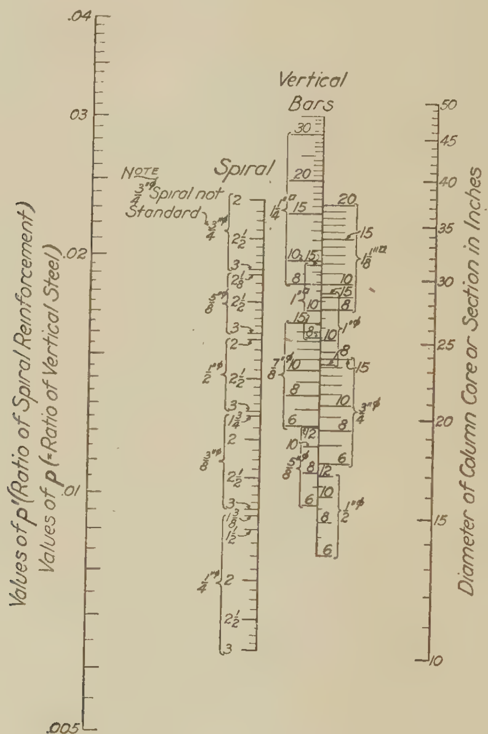
DIAGRAM 98.—DESIGN OF SPIRAL COLUMNS—5000-LB. CONCRETE



See instructions for use under Diagram 100.

$p = .04$ when $\frac{P}{A} = 1920$. (See Section 1103–b of the code.)

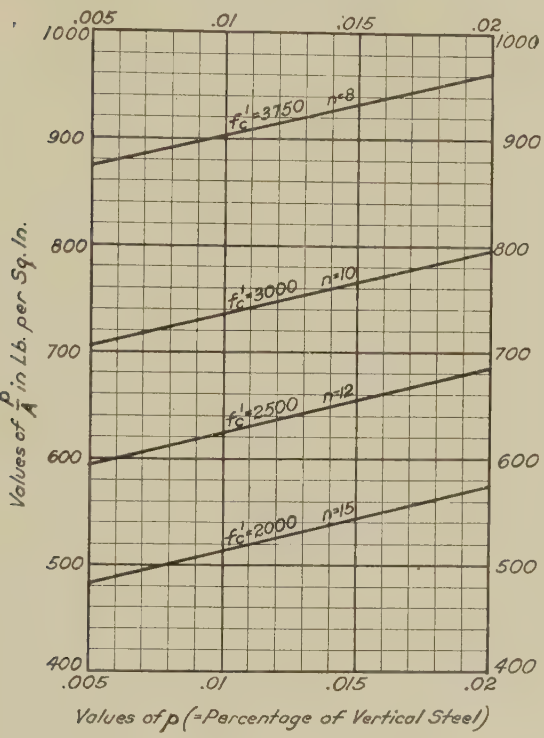
DIAGRAM 99.—SELECTION CHART FOR COLUMN REINFORCEMENT



INSTRUCTIONS FOR USE.—This diagram is really two diagrams superimposed for convenience—a spiral selection diagram made up of the left, left-center and right scales and a vertical bar selection diagram made up to the left, right-center and right scales. It is to be used in connection with Diagram 100 to select vertical bars in *round* tied columns and in the design of composite columns

This is an alignment chart, read by the aid of a straight edge, preferably the edge of a celluloid triangle on account of its transparency. Set the straight edge on the right hand scale to the diameter of the effective column section and on the left scale to the required percentage of *spiral* reinforcement and read off the spiral wire and pitch at the intersection of the straight edge with the *left-center* scale. For the vertical bars, set the straight edge on the right hand scale as before and on the left hand scale to the percentage of *vertical* reinforcement. Read off the number and size of vertical bars at the intersection of the straight edge with the *right-center* scale.

DIAGRAM 100.—DESIGN OF TIED COLUMNS
2000-LB., 2500-LB., 3000-LB. AND 3750-LB. CONCRETE



INSTRUCTIONS FOR USE.—The area A is the full area of the column section including the fireproofing, but the vertical bars are required to be set 2 inches in the clear from the surface.

The minimum vertical steel permitted is 4-⅝ in. rd. bars. The ties must be at least ¼ in. rd. in size, spaced not over 12 inches on centers, and must be so arranged as to afford support to each vertical bar in at least two directions, as called for by Section 1104 of the code.

GENERAL INSTRUCTIONS FOR USE OF DIAGRAMS 94 TO 98

This is an alignment chart, read by the aid of a straight edge, preferably the edge of a celluloid triangle on account of its transparency. Set the edge on the right hand scale to the diameter of the column core and on the left hand scale to the value of P/A (stress in lb. per sq. in. of core area). Read off the *spiral* wire and pitch at the intersection of the straight edge with the *left-center* scale and the number and size of *longitudinal* bars at the intersection with the *right-center* scale. A is found from Table 105.

TABLE 101.—PERCENTAGES AND WEIGHTS OF ¼-IN. ROUND SPIRALS

	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
10.....	0.66	0.69	0.72	0.75	0.79	0.83	0.87	0.92	0.98	1.05	1.12	1.21	1.31
	1.75	**1.83**	**1.91**	**2.00**	**2.10**	**2.21**	**2.33**	**2.47**	**2.62**	**2.80**	**3.00**	**3.23**	**3.50**
11.....	0.60	0.62	0.65	0.68	0.71	0.75	0.79	0.84	0.89	0.95	1.02	1.10	1.19
	1.92	**2.01**	**2.10**	**2.20**	**2.31**	**2.43**	**2.56**	**2.72**	**2.88**	**3.08**	**3.30**	**3.55**	**3.85**
12.....	0.55	0.57	0.59	0.62	0.65	0.69	0.73	0.77	0.82	0.87	0.93	1.01	1.09
	2.10	**2.19**	**2.29**	**2.40**	**2.52**	**2.65**	**2.80**	**2.96**	**3.15**	**3.36**	**3.60**	**3.88**	**4.19**
13.....	0.51	0.53	0.55	0.58	0.61	0.64	0.67	0.71	0.75	0.81	0.87	0.93	1.00
	2.27	**2.38**	**2.48**	**2.60**	**2.73**	**2.88**	**3.03**	**3.21**	**3.41**	**3.64**	**3.90**	**4.20**	**4.54**
14.....	0.47	0.49	0.51	0.53	0.56	0.59	0.63	0.66	0.70	0.75	0.80	0.86	0.93
	2.45	**2.56**	**2.67**	**2.80**	**2.94**	**3.10**	**3.26**	**3.46**	**3.67**	**3.92**	**4.20**	**4.52**	**4.90**
15.....	0.44	0.46	0.48	0.50	0.52	0.55	0.58	0.62	0.66	0.70	0.75	0.81	0.87
	2.62	**2.74**	**2.87**	**3.00**	**3.15**	**3.32**	**3.50**	**3.71**	**3.94**	**4.20**	**4.50**	**4.85**	**5.24**
16.....	0.41	0.43	0.45	0.47	0.49	0.52	0.55	0.58	0.61	0.65	0.70	0.76	0.82
	2.80	**2.92**	**3.05**	**3.20**	**3.36**	**3.54**	**3.73**	**3.95**	**4.20**	**4.48**	**4.80**	**5.17**	**5.60**
17.....	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.58	0.62	0.66	0.71	0.77
	2.97	**3.10**	**3.24**	**3.40**	**3.56**	**3.75**	**3.96**	**4.19**	**4.46**	**4.76**	**5.10**	**5.49**	**5.94**
18.....	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.58	0.62	0.67	0.73
	3.15	**3.29**	**3.44**	**3.60**	**3.78**	**3.98**	**4.20**	**4.45**	**4.73**	**5.05**	**5.40**	**5.82**	**6.28**
19.....	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.59	0.64	0.69
	3.32	**3.47**	**3.62**	**3.80**	**3.98**	**4.20**	**4.43**	**4.69**	**4.98**	**5.32**	**5.70**	**6.14**	**6.64**
20.....	0.33	0.34	0.36	0.38	0.40	0.42	0.44	0.46	0.49	0.52	0.56	0.60	0.65
	3.50	**3.65**	**3.82**	**4.00**	**4.20**	**4.42**	**4.66**	**4.93**	**5.25**	**5.60**	**6.00**	**6.46**	**7.00**
21.....	0.31	0.32	0.34	0.36	0.38	0.40	0.42	0.44	0.47	0.50	0.53	0.57	0.62
	3.67	**3.84**	**4.01**	**4.20**	**4.40**	**4.64**	**4.90**	**5.18**	**5.51**	**5.88**	**6.30**	**6.78**	**7.34**
22.....	0.30	0.31	0.32	0.34	0.36	0.38	0.40	0.42	0.45	0.48	0.51	0.55	0.60
	3.85	**4.02**	**4.20**	**4.40**	**4.62**	**4.86**	**5.13**	**5.43**	**5.77**	**6.16**	**6.60**	**7.11**	**7.70**
23.....	0.29	0.30	0.31	0.32	0.34	0.36	0.38	0.40	0.43	0.46	0.49	0.53	0.57
	4.02	**4.20**	**4.38**	**4.60**	**4.83**	**5.08**	**5.36**	**5.67**	**6.03**	**6.44**	**6.90**	**7.43**	**8.05**
24.....	0.28	0.29	0.30	0.31	0.33	0.35	0.37	0.39	0.41	0.44	0.47	0.51	0.55
	4.20	**4.38**	**4.57**	**4.80**	**5.04**	**5.30**	**5.60**	**5.92**	**6.30**	**6.72**	**7.20**	**7.75**	**8.40**
25.....	0.26	0.27	0.28	0.29	0.31	0.33	0.35	0.37	0.39	0.42	0.45	0.48	0.52
	4.37	**4.57**	**4.78**	**5.00**	**5.25**	**5.53**	**5.83**	**6.17**	**6.56**	**7.00**	**7.50**	**8.07**	**8.75**
26.....	0.25	0.26	0.27	0.28	0.30	0.32	0.34	0.36	0.38	0.40	0.43	0.46	0.50
	4.55	**4.75**	**4.96**	**5.20**	**5.46**	**5.75**	**6.06**	**6.42**	**6.82**	**7.28**	**7.80**	**8.39**	**9.10**
27.....	0.24	0.25	0.26	0.27	0.29	0.30	0.32	0.34	0.36	0.38	0.41	0.44	0.48
	4.72	**4.94**	**5.15**	**5.40**	**5.68**	**5.96**	**6.30**	**6.66**	**7.08**	**7.56**	**8.10**	**8.72**	**9.45**
28.....	0.23	0.24	0.25	0.26	0.27	0.29	0.31	0.33	0.35	0.37	0.40	0.43	0.47
	4.90	**5.11**	**5.34**	**5.60**	**5.88**	**6.19**	**6.53**	**6.91**	**7.34**	**7.84**	**8.40**	**9.04**	**9.80**
29.....	0.23	0.24	0.25	0.26	0.27	0.28	0.30	0.32	0.34	0.36	0.39	0.42	0.45
	5.07	**5.30**	**5.53**	**5.80**	**6.09**	**6.41**	**6.77**	**7.16**	**7.60**	**8.12**	**8.70**	**9.36**	**10.15**

INSTRUCTIONS FOR USE OF TABLES 101 TO 104.—The figures in light face type represent the percentage of spiral reinforcement afforded by a spiral wire of the size heading the table at the pitch shown at the top of each column when placed about a core of the diameter (in inches) appearing at the left. The figures in heavy face type represent the weight per foot of column of the percentage of spiral directly above. This weight includes only the wire. Allowance must be made for the extra wire to finish a spiral unit securely at its ends and for the weight of the three or more spacers. Each ⅞-in. channel spacer adds ¾ pound per foot of spiral. Different manufacturers use different spacers and the weight of this item will vary accordingly.

TABLE 102.—PERCENTAGES AND WEIGHTS OF ⅜-IN. ROUND SPIRALS

	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
15.....	0.98	1.02	1 07	1.12	1.18	1.24	1.31	1.39	1.47	1.57	1.68	1.82	1.96
	5.91	6.16	6.44	6.73	7.08	7.46	7.87	8.32	8.85	9.4[illegible]	10.1	10.9	11.8
16.....	0.92	0.96	1.00	1.05	1.10	1.16	1.23	1.30	1.38	1.47	1.58	1.70	1.84
	6.30	6.57	6.87	7.18	7.55	7.95	8.39	8.88	9.44	10.1	10.8	11.6	12.6
17.....	0.87	0.90	0.94	0.99	1.04	1.09	1.15	1.22	1.30	1.39	1.48	1.60	1 73
	6.69	6.98	7.30	7.63	8.02	8.45	8.92	9.44	10.0	10.7	11.5	12.3	13.4
18.....	0.82	0.85	0.89	0.93	0.98	1.03	1.09	1.16	1.23	1.31	1.40	1.51	1.64
	7.08	7.39	7.73	8.08	8.50	8.94	9.44	10.0	10.6	11.3	12.1	13.1	14.2
19.....	0.78	0.81	0.84	0 88	0.93	0.98	1.03	1.09	1.16	1.24	1.33	1.43	1.55
	7.48	7.81	8.15	8.53	8.97	9.45	9.97	10.5	11.2	12.0	12.8	13.8	14.9
20.....	0.74	0.77	0.80	0.84	0.88	0.93	0.98	1.04	1.10	1.18	1.26	1.36	1.47
	7 87	8.21	8.59	8.98	9.44	9.94	10.5	11.1	11.8	12.6	13.5	14.5	15.7
21.....	0.70	0.73	0.76	0.80	0.84	0.88	0.94	0.99	1.05	1.12	1.20	1.29	1.40
	8.26	8.62	9.02	9.43	9.92	10.4	11.0	11.6	12.4	13.2	14.2	15.2	16.5
22.....	0.67	0.70	0.73	0.76	0.80	0.84	0.89	0.94	1.00	1.07	1.15	1.23	1.34
	8.66	9.04	9.45	9.88	10.4	10.9	11.5	12.2	13.0	13.8	14.8	16.0	17.3
23.....	0.64	0.67	0.70	0.73	0.77	0.81	0.85	0.90	0.96	1.02	1.10	1.18	1.28
	9.05	9.45	9.88	10.3	10.9	11.4	12.1	12.8	13.6	14.5	15.5	16.7	18.1
24.....	0.61	0.64	0.67	0.70	0.73	0.77	0.82	0.87	0.92	0.98	1.05	1.13	1.23
	9.45	9.86	10.3	10.8	11.3	11.9	12.6	13.3	14.2	15.1	16.2	17.4	18.9
25.....	0.59	0.61	0.64	0.67	0.70	0.74	0.78	0.83	0.88	0.94	1.01	1.09	1.18
	9.84	10.3	10.7	11.2	11.8	12.4	13.1	13.9	14.7	15.7	16.9	18.2	19.7
26.....	0.57	0.59	0.62	0.65	0.68	0.71	0.75	0.80	0.85	0.90	0.97	1.05	1.13
	10.2	10.7	11.2	11.7	12.3	12.9	13.6	14.4	15.4	16.4	17.5	18.9	20.4
27.....	0.55	0.57	0.59	0.62	0.65	0.69	0.73	0.77	0.82	0.87	0.94	1.01	1.09
	10.6	11.1	11.6	12.1	12.7	13.4	14.2	15.0	15.9	17.0	18.2	19.6	21.2
28.....	0.53	0.55	0.57	0.60	0.63	0.66	0.70	0.74	0.79	0.84	0.90	0.97	1.05
	11.0	11.5	12.0	12.6	13.2	13.9	14.7	15.5	16.5	17.6	18.9	20.4	22.0
29.....	0.51	0.53	0.55	0.58	0.61	0.64	0.68	0.72	0.76	0.81	0.87	0.94	1.01
	11.4	11.9	12.5	13.0	13.7	14.4	15.2	16.1	17.1	18.3	19.6	21.1	22.8
30.....	0.49	0.51	0.53	0.56	0.59	0.62	0.65	0.69	0.74	0.79	0.84	0.91	0.98
	11.8	12.3	12.9	13.5	14.2	14.9	15.7	16.6	17.7	18.9	20.2	21.8	23.6
31.....	0.48	0.50	0.52	0.54	0.57	0.60	0.63	0.67	0.72	0.76	0.82	0.88	0.95
	12.2	12.7	13.3	13.9	14.6	15.4	16.3	17.2	18.3	19.5	20.9	22.5	24.4
32.....	0.46	0.48	0.50	0.52	0.55	0.58	0.61	0.65	0.69	0.74	0.79	0.85	0.92
	12.6	13.1	13.7	14.4	15.1	15.9	16.8	17.8	18.9	20.1	21.6	23.3	25.2
33.....	0.45	0.47	0.49	0.51	0.54	0.57	0.60	0.63	0.67	0.71	0.77	0.83	0.89
	13.0	13.5	14.2	14.8	15.6	16.4	17.3	18.3	19.5	20.8	22.3	24.0	26.0
34.....	0.44	0.45	0.47	0.49	0.52	0.55	0.58	0.61	0.65	0.70	0.75	0.81	0.87
	13.4	14.0	14.6	15.3	16.1	16.9	17.8	18.9	20.1	21.4	23.0	24.7	26.8
35.....	0.42	0.44	0.46	0.48	0.50	0.53	0.56	0.59	0.63	0.67	0.72	0.78	0.84
	13.8	14.4	15.0	15.7	16.5	17.4	18.4	19.4	20.7	22.0	23.6	25.4	27.5
36.....	0.41	0.43	0.45	0.47	0.49	0.52	0.55	0.58	0.61	0.65	0.70	0.75	0.82
	14.2	14.8	15.4	16.2	17.0	17.9	18.9	20.0	21.3	22.7	24.3	26.2	28.3
37.....	0.40	0.41	0.43	0.45	0.47	0.50	0.53	0.56	0.60	0.64	0.68	0.73	0.80
	14.6	15.2	15.9	16.6	17.5	18.4	19.4	20.6	21.9	23.3	25.0	26.9	29.1
38.....	0.39	0.40	0.42	0.44	0.46	0.49	0.52	0.55	0.58	0.62	0.66	0.71	0.77
	15.0	15.6	16.3	17.1	18.0	18.9	19.9	21.1	22.4	23.9	25.6	27.6	29.9
39.....	0.38	0.39	0.41	0.43	0.45	0.47	0.50	0.53	0.57	0.61	0.65	0.70	0.76
	15.4	16.0	16.7	17.5	18.4	19.4	20.5	21.6	23.0	24.6	26.3	28.3	30.7

See instructions for use under Table 101.

TABLE 103.—PERCENTAGES AND WEIGHTS OF ½-IN. ROUND SPIRALS

	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
15.....	1.75	1.82	1.90	1.99	2.09	2.21							
	10.5	**11.0**	**11.4**	**12.0**	**12.6**	**13.2**							
16.....	1.63	1.70	1.78	1.87	1.96	2.07							
	11.2	**11.7**	**12.2**	**12.8**	**13.4**	**14.1**							
17.....	1.54	1.61	1.68	1.76	1.85	1.95	2.05						
	11.9	**12.4**	**13.0**	**13.6**	**14.3**	**15.0**	**15.9**						
18.....	1.45	1.52	1.59	1.66	1.75	1.84	1.94	2.06					
	12.6	**13.1**	**13.7**	**14.4**	**15.1**	**15.9**	**16.8**	**17.8**					
19.....	1.38	1.44	1.50	1.57	1.65	1.74	1.84	1.95	2.07				
	13.3	**13.9**	**14.5**	**15.2**	**16.0**	**16.8**	**17.7**	**18.8**	**20.0**				
20.....	1.30	1.36	1.43	1.50	1.57	1.65	1.74	1.84	1.96	2.09			
	14.0	**14.6**	**15.3**	**16.0**	**16.8**	**17.7**	**18.6**	**19.8**	**21.0**	**22.4**			
21.....	1.25	1.30	1.36	1.43	1.50	1.57	1.66	1.76	1.87	2.00			
	14.7	**15.3**	**16.0**	**16.8**	**17.6**	**18.5**	**19.6**	**20.8**	**22.0**	**23.5**			
22.....	1.19	1.24	1.30	1.36	1.43	1.50	1.59	1.68	1.79	1.90	2.04		
	15.4	**16.1**	**16.8**	**17.6**	**18.5**	**19.4**	**20.5**	**21.7**	**23.1**	**24.6**	**26.4**		
23.....	1.14	1.19	1.24	1.30	1.37	1.44	1.52	1.60	1.71	1.82	1.95	2.10	
	16.1	**16.8**	**17.6**	**18.4**	**19.3**	**20.3**	**21.4**	**22.7**	**24.1**	**25.8**	**27.6**	**29.7**	
24.....	1.09	1.14	1.19	1.25	1.31	1.38	1.45	1.54	1.64	1.75	1.87	2.02	
	16.8	**17.5**	**18.3**	**19.2**	**20.2**	**21.2**	**22.4**	**23.7**	**25.2**	**26.9**	**28.8**	**31.0**	
25.....	1.05	1.09	1.14	1.20	1.26	1.32	1.40	1.48	1.57	1.67	1.80	1.94	2.10
	17.5	**18.3**	**19.1**	**20.0**	**21.0**	**22.1**	**23.3**	**24.7**	**26.2**	**28.0**	**30.0**	**32.3**	**35.0**
26.....	1.01	1.05	1.10	1.15	1.21	1.27	1.34	1.42	1.51	1.60	1.73	1.86	2.01
	18.2	**19.0**	**19.9**	**20.8**	**21.8**	**23.0**	**24.3**	**25.7**	**27.2**	**29.1**	**31.2**	**33.6**	**36.4**
27.....	0.97	1.01	1.06	1.11	1.16	1.22	1.29	1.37	1.45	1.55	1.66	1.78	1.94
	18.9	**19.7**	**20.6**	**21.6**	**22.7**	**23.9**	**25.2**	**26.7**	**28.3**	**30.2**	**32.4**	**34.9**	**37.8**
28.....	0.93	0.97	1.02	1.07	1.12	1.18	1.25	1.32	1.40	1.49	1.60	1.72	1.87
	19.6	**20.4**	**21.4**	**22.4**	**23.5**	**24.7**	**26.1**	**27.7**	**29.4**	**31.3**	**33.6**	**36.2**	**39.2**
29.....	0.90	0.94	0.98	1.03	1.08	1.14	1.20	1.27	1.35	1.44	1.54	1.66	1.80
	20.3	**21.2**	**22.2**	**23.2**	**24.4**	**25.6**	**27.1**	**28.7**	**30.4**	**32.5**	**34.8**	**37.5**	**40.6**
30.....	0.87	0.91	0.95	1.00	1.05	1.10	1.16	1.23	1.31	1.40	1.50	1.61	1.74
	21.0	**21.9**	**22.9**	**24.0**	**25.2**	**26.5**	**28.0**	**29.6**	**31.5**	**33.6**	**36.0**	**38.8**	**42.0**
31.....	0.84	0.88	0.92	0.96	1.01	1.07	1.13	1.19	1.27	1.35	1.45	1.56	1.68
	21.7	**22.6**	**23.6**	**24.8**	**26.0**	**27.4**	**28.9**	**30.6**	**32.5**	**34.7**	**37.2**	**40.0**	**43.3**
32.....	0.82	0.85	0.89	0.93	0.98	1.03	1.09	1.16	1.23	1.31	1.41	1.51	1.64
	22.4	**23.4**	**24.4**	**25.6**	**26.8**	**28.3**	**29.8**	**31.6**	**33.6**	**35.8**	**38.4**	**41.3**	**44.7**
33.....	0.79	0.83	0.87	0.91	0.95	1.00	1.06	1.12	1.19	1.27	1.36	1.46	1.59
	23.1	**24.1**	**25.2**	**26.4**	**27.7**	**29.2**	**30.8**	**32.6**	**34.6**	**36.9**	**39.6**	**42.6**	**46.1**
34.....	0.77	0.80	0.84	0.88	0.92	0.97	1.03	1.09	1.16	1.24	1.32	1.42	1.54
	23.8	**24.8**	**25.9**	**27.2**	**28.5**	**30.0**	**31.7**	**33.5**	**35.6**	**38.0**	**40.8**	**43.9**	**47.6**
35.....	0.75	0.78	0.81	0.85	0.90	0.95	1.00	1.05	1.12	1.19	1.28	1.38	1.50
	24.5	**25.6**	**26.7**	**28.0**	**29.4**	**30.9**	**32.6**	**34.5**	**36.7**	**39.2**	**42.0**	**45.2**	**48.9**
36.....	0.73	0.76	0.79	0.83	0.87	0.92	0.97	1.02	1.09	1.16	1.25	1.35	1.45
	25.2	**26.3**	**27.5**	**28.8**	**30.2**	**31.8**	**33.6**	**35.5**	**37.8**	**40.3**	**43.2**	**46.5**	**50.4**
37.....	0.71	0.74	0.77	0.81	0.85	0.89	0.94	1.00	1.06	1.13	1.21	1.30	1.41
	25.9	**27.0**	**28.2**	**29.6**	**31.1**	**32.7**	**34.5**	**36.5**	**38.8**	**41.4**	**44.4**	**47.8**	**51.8**
38.....	0.69	0.72	0.75	0.79	0.83	0.87	0.92	0.97	1.03	1.10	1.18	1.27	1.38
	26.6	**27.8**	**29.1**	**30.4**	**31.9**	**33.6**	**35.5**	**37.5**	**39.9**	**42.5**	**45.6**	**49.1**	**53.2**
39.....	0.67	0.70	0.73	0.77	0.81	0.85	0.90	0.95	1.01	1.07	1.15	1.24	1.34
	27.3	**28.5**	**29.7**	**31.2**	**32.8**	**34.5**	**36.4**	**38.5**	**40.9**	**43.6**	**46.8**	**50.4**	**54.6**

See instructions for use under Table 101.

TABLE 104.—PERCENTAGES AND WEIGHTS OF ⅝-IN. ROUND SPIRALS

	PITCH												
	3 in.	2⅞ in.	2¾ in.	2⅝ in.	2½ in.	2⅜ in.	2¼ in.	2⅛ in.	2 in.	1⅞ in.	1¾ in.	1⅝ in.	1½ in.
30.....	1.36	1.42	1.49	1.56	1.64	1.72	1.82	1.93	2.05				
	32.8	34.3	35.8	37.5	39.4	41.5	43.8	46.4	49.3				
31.....	1.32	1.38	1.44	1.51	1.59	1.67	1.76	1.87	1.98				
	33.9	35.4	37.0	38.8	40.8	42.9	45.3	47.9	50.9				
32.....	1.28	1.33	1.39	1.46	1.54	1.62	1.71	1.81	1.92	2.04			
	35.1	36.5	38.2	40.1	42.2	44.3	46.7	49.5	52.6	56.1			
33.....	1.24	1.30	1.35	1.42	1.49	1.57	1.66	1.75	1.86	1.99			
	36.1	37.7	39.4	41.3	43.3	45.6	48.2	51.1	54.2	57.8			
34.....	1.21	1.26	1.32	1.38	1.45	1.52	1.61	1.70	1.81	1.93			
	37.3	38.8	40.6	42.6	44.7	47.1	49.7	52.6	55.9	59.6			
35.....	1.17	1.22	1.28	1.34	1.41	1.48	1.56	1.65	1.76	1.87	2.01		
	38.3	40.0	41.8	43.8	46.1	48.4	51.2	54.2	57.5	61.3	65.7		
36.....	1.14	1.19	1.24	1.30	1.37	1.44	1.52	1.61	1.71	1.82	1.95		
	39.4	41.2	42.9	45.0	47.4	49.8	52.6	55.7	59.2	63.1	67.6		
37.....	1.11	1.16	1.21	1.27	1.33	1.40	1.48	1.56	1.66	1.77	1.90	2.04	
	40.5	42.3	44.2	46.3	48.6	51.2	54.1	57.2	60.8	64.8	69.5	74.8	
38.....	1.08	1.13	1.18	1.23	1.29	1.36	1.44	1.52	1.62	1.73	1.85	1.99	
	41.6	43.4	45.3	47.6	49.9	52.6	55.5	58.8	62.4	66.7	71.3	76.8	
39.....	1.05	1.10	1.15	1.20	1.26	1.33	1.40	1.48	1.58	1.68	1.80	1.94	
	42.7	44.6	46.6	48.8	51.2	53.9	56.9	60.3	64.1	68.3	73.2	78.8	
40.....	1.02	1.07	1.12	1.17	1.23	1.29	1.36	1.45	1.54	1.64	1.76	1.89	2.05
	43.8	45.7	47.8	50.1	52.6	55.3	58.4	61.9	65.8	70.1	75.2	80.8	87.6
41.....	1.00	1.04	1.09	1.14	1.20	1.26	1.33	1.41	1.50	1.60	1.71	1.85	2.00
	44.9	46.8	48.9	51.3	53.8	56.7	59.9	63.3	67.4	71.8	76.9	83.8	89.8
42.....	0.98	1.02	1.06	1.12	1.17	1.23	1.30	1.38	1.46	1.56	1.67	1.80	1.95
	46.0	47.9	49.9	52.6	55.2	58.1	61.3	65.0	69.1	73.6	78.8	84.8	92.0
43.....	0.95	1.00	1.04	1.09	1.14	1.20	1.27	1.35	1.43	1.53	1.63	1.76	1.91
	47.0	49.2	51.3	53.8	56.5	59.4	62.8	66.5	70.7	75.3	80.7	86.9	94.2
44.....	0.93	0.98	1.02	1.07	1.12	1.18	1.24	1.32	1.40	1.49	1.60	1.72	1.87
	48.2	50.3	52.5	55.1	57.8	60.9	64.2	68.2	72.3	77.1	82.6	88.9	96.5
45.....	0.91	0.95	0.99	1.04	1.09	1.15	1.21	1.29	1.37	1.46	1.56	1.68	1.82
	49.2	51.4	53.7	56.3	59.1	62.2	65.7	69.6	74.0	78.9	84.5	90.9	98.6
46.....	0.89	0.93	0.97	1.02	1.07	1.13	1.19	1.26	1.34	1.43	1.53	1.65	1.78
	50.3	52.6	54.8	57.6	60.4	63.7	67.2	71.2	75.7	80.7	86.4	93.0	100.7
47.....	0.87	0.91	0.95	1.00	1.05	1.10	1.16	1.23	1.31	1.40	1.50	1.61	1.74
	51.4	53.7	56.1	58.8	61.8	65.0	68.7	72.7	77.3	82.4	88.3	95.0	102.8
48.....	0.85	0.89	0.93	0.98	1.02	1.08	1.14	1.20	1.28	1.36	1.46	1.57	1.71
	52.6	54.8	57.3	60.1	63.1	66.4	70.1	74.2	78.9	84.1	90.1	97.0	105.1
49.....	0.84	0.87	0.91	0.96	1.00	1.06	1.12	1.18	1.25	1.34	1.44	1.54	1.68
	53.7	55.9	58.4	61.3	64.4	67.8	71.6	75.7	80.6	85.9	92.0	99.1	107.3

See instructions for use under Table 101.

For spirals of heavier wire or larger diameter than are included in Tables 101 to 104, the percentage may computed approximately by the formula:

$$p' = \frac{4a_s}{(d)\ (\text{pitch})}$$

in which p' = percentage of area of circle of diameter, d.
a_s = cross sectional area of spiral rod.
d = diameter of spiral (generally listed as overall or outside diameter).
pitch = distance between centers of successive turns measured parallel to axis of spiral.

TABLE 105.—PERIMETERS, VOLUMES AND CORE AREAS FOR ROUND COLUMNS

Diameter		Core Area, sq. in.	Rd. Column Perimeter		Volume, cu. ft. per ft.		
Column, in.	Core, in.		Ft.	In.	Round	Octagonal	Square
14	10	78.5	3	8	1.07	1.13	1.36
	11	95.0					
16	12	113.1	4	2	1.40	1.47	1.78
	13	132.7					
18	14	153.9	4	9	1.77	1.86	2.25
	15	176.7					
20	16	201.0	5	3	2.18	2.30	2.78
	17	227.0					
22	18	254.5	5	9	2.64	2.78	3.36
	19	283.5					
24	20	314.2	6	3	3.14	3.31	4.00
	21	346.4					
26	22	380.1	6	10	3.69	3.89	4.69
	23	415.4					
28	24	452.4	7	4	4.28	4.51	5.44
	25	490.9					
30	26	530.9	7	10	4.91	5.18	6.25
	27	572.6					
32	28	615.8	8	5	5.58	5.89	7.12
	29	660.5					
34	30	706.9	8	11	6.30	6.55	8.02
	31	754.8					
36	32	804.2	9	5	7.07	7.46	9.00
	33	855.3					
38	34	907.9	10	0	7.88	8.31	10.03
	35	962.1					
40	36	1,017.9	10	6	8.73	9.21	11.11
	37	1,075.2					

TABLE 106.—VOLUMES OF FLAT SLAB COLUMN SHAFTS AND CAPITALS

Round Columns								Square Columns							
Diameter of Column Shaft	Volume of Column Shaft	Diameter of Column Capital						Side of Column Shaft	Volume of Column Shaft	Side of Column Capital					
		3 ft. 6 in.	4 ft. 0 in.	4 ft. 6 in.	5 ft. 0 in.	5 ft. 6 in.	6 ft. 0 in.			3 ft. 6 in.	4 ft. 0 in.	4 ft. 6 in.	5 ft. 0 in.	5 ft. 6 in.	6 ft. 0 in.
14	1.07	5.23	....	....	....	....	....	14	1.36	6.65	....	....	....	....	....
16	1.40	4.81	7.61	....	....	....	....	16	1.78	6.13	9.67	....	....	....	....
18	1.77	4.38	7.07	10.58	....	....	....	18	2.25	5.59	9.01	13.49	....	....	....
20	2.18	3.93	6.52	9.94	14.30	....	....	20	2.78	5.01	8.30	12.66	18.21	....	....
22	2.64	3.48	5.95	9.25	13.48	18.75	....	22	3.36	4.43	7.57	11.79	17.18	23.90	....
24	3.14	3.02	5.36	8.54	12.66	17.80	24.08	24	4.00	3.85	6.83	10.88	16.12	22.68	30.67
26	3.69	2.55	4.78	7.83	11.80	16.82	22.96	26	4.69	3.26	6.09	9.96	15.03	21.41	29.23
28	4.28	2.13	4.18	7.08	10.91	15.78	21.76	28	5.44	2.71	5.33	9.03	13.90	20.09	27.71
30	4.91	1.70	3.60	6.34	10.00	14.72	20.54	30	6.25	2.16	4.59	8.08	12.75	18.75	26.17
32	5.58	....	3.04	5.60	9.12	13.66	19.31	32	7.11	....	3.87	7.14	11.61	17.40	24.60
34	6.30	....	2.48	4.89	8.21	12.58	18.05	34	8.03	....	3.18	6.23	10.46	16.01	23.00
36	7.07	....	2.01	4.20	7.32	11.50	16.78	36	9.00	....	2.51	5.35	9.32	14.66	21.38
38	7.88	....	....	3.53	6.44	10.40	15.51	38	10.02	....	....	4.50	8.20	13.27	19.76
40	8.73	....	....	2.88	5.60	9.36	14.24	40	11.11	....	....	3.67	7.13	11.91	18.13
42	9.62	....	....	2.26	4.78	8.31	12.96	42	12.25	....	....	2.90	6.09	10.59	16.51
44	10.56	....	....	....	4.00	7.29	11.71	44	13.44	....	....	....	5.09	9.29	14.92
46	11.54	....	....	....	3.24	6.31	10.48	46	14.69	....	....	....	4.14	8.04	13.35
48	12.57	....	....	....	2.55	5.36	9.29	48	16.00	....	....	....	3.25	6.84	11.82

See note under Table 107, next page.

TABLE 107.—AREAS AND PERIMETERS OF ROUND SECTIONS

Diameter		Area		Perimeter	Diameter		Area		Perimeter
Ft. In.	In.	Sq. In.	Sq. Ft.	Ft.	Ft. In.	In.	Sq. In.	Sq. Ft.	Ft.
1-1	13	132.7	0.92	3.40	4-7	55	2,376	16.50	14.40
1-1	14	153.9	1.07	3.66	4-8	56	2,463	17.10	14.66
1-3	15	176.7	1.23	3.93	4-9	57	2,552	17.72	14.92
1-4	16	201.1	1.40	4.18	4-10	58	2,642	18.35	15.18
1-5	17	227.0	1.58	4.45	4-11	59	2,734	18.99	15.45
1-6	18	254.5	1.77	4.71	5-0	60	2,827	19.63	15.71
1-7	19	283.5	1.97	4.97	5-1	61	2,922	20.29	15.97
1-8	20	314.2	2.18	5.23	5-2	62	3,019	20.97	16.23
1-9	21	346.4	2.41	5.50	5-3	63	3,117	21.65	16.49
1-10	22	380.1	2.64	5.76	5-4	64	3,217	22.34	16.76
1-11	23	415.5	2.89	6.02	5-5	65	3,318	23.04	17.02
2-0	24	452.4	3.14	6.28	5-6	66	3,421	23.76	17.28
2-1	25	490.9	3.41	6.55	5-7	67	3,526	24.48	17.54
2-2	26	530.9	3.69	6.81	5-8	68	3,632	25.22	17.80
2-3	27	572.6	3.98	7.07	5-9	69	3,739	25.97	18.06
2-4	28	615.8	4.28	7.33	5-10	70	3,848	26.72	18.32
2-5	29	660.5	4.59	7.58	5-11	71	3,959	27.49	18.58
2-6	30	706.9	4.91	7.86	6-0	72	4,072	28.27	18.85
2-7	31	754.8	5.24	8.12	6-1	73	4,185	29.06	19.11
2-8	32	804.2	5.58	8.38	6-2	74	4,301	29.87	19.37
2-9	33	855.3	5.94	8.64	6-3	75	4,418	30.68	19.63
2-10	34	907.9	6.30	8.89	6-4	76	4,536	31.50	19.90
2-11	35	962.1	6.68	9.16	6-5	77	4,657	32.34	20.16
3-0	36	1,017.9	7.07	9.42	6-6	78	4,778	33.18	20.42
3-1	37	1,075.2	7.47	9.68	6-7	79	4,902	34.04	20.68
3-2	38	1,134.1	7.88	9.95	6-8	80	5,027	34.91	20.94
3-3	39	1,194.6	8.30	10.21	6-9	81	5,153	35.78	21.21
3-4	40	1,256.6	8.73	10.47	6-10	82	5,281	36.67	21.47
3-5	41	1,320.3	9.17	10.72	6-11	83	5,411	37.58	21.73
3-6	42	1,385.4	9.62	10.99	7-0	84	5,542	38.48	21.99
3-7	43	1,452.2	10.08	11.26	7-1	85	5,674	39.40	22.25
3-8	44	1,520.5	10.56	11.52	7-2	86	5,809	40.34	22.51
3-9	45	1,590.4	11.04	11.79	7-3	87	5,945	41.28	22.78
3-10	46	1,661.9	11.54	12.04	7-4	88	6,082	42.24	23.04
3-11	47	1,734.9	12.05	12.31	7-5	89	6,221	43.20	23.30
4-0	48	1,809.6	12.57	12.57	7-6	90	6,362	44.18	23.56
4-1	49	1,885.7	13.10	12.83	7-7	91	6,504	45.17	23.82
4-2	50	1,963.5	13.64	13.10	7-8	92	6,648	46.16	24.09
4-3	51	2,042.8	14.19	13.35	7-9	93	6,793	47.17	24.35
4-4	52	2,123.7	14.75	13.61	7-10	94	6,940	48.19	24.61
4-5	53	2,206.2	15.32	13.88	7-11	95	7,088	49.22	24.87
4-6	54	2,290.2	15.90	14.14	8-0	96	7,238	50.27	25.13

INSTRUCTIONS FOR USE OF TABLE 106. For octagonal columns and capitals add 5½ per cent to values given for round columns and capitals. Volume of concrete in column shafts is given in cubic feet per foot of height. For column capitals it is given in cubic feet and includes only the concrete in the capital outside the surface of the column enclosed in the capital.

PROPORTIONS ADOPTED FOR DESIGN DIAGRAMS FOR SQUARE FOOTINGS.

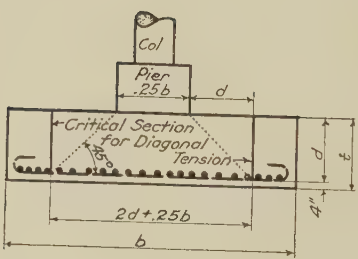


FIG. 16.—FLAT-TOP FOOTINGS.

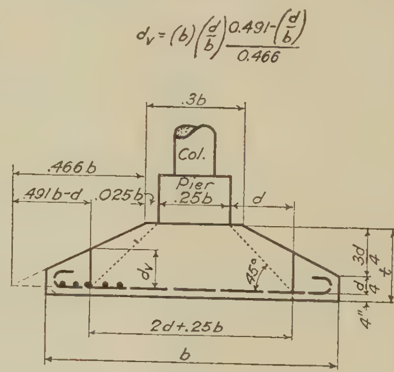


FIG. 17.—SLOPED-TOP FOOTINGS.

INSTRUCTIONS FOR USE OF DIAGRAM 108

This diagram applies only to footings of the standardized proportions shown in Figs. 16 and 17. If the footing bars are straight and extend to within 2 inches of the edges of the footing the value of v_c is 0.02 f'_c. If the footing bars are provided with special anchorage the total length of the hooked bar being the overall width of the footing plus 20 bar diameters, the value of v_c may be taken as 0.03 f'_c. The value of w is the load at the top of the footing divided by the area of the base of the footing. d is the effective depth of footing to the reinforcing steel and b the side of the square base of the footing.

INSTRUCTIONS FOR USE OF DIAGRAM 109

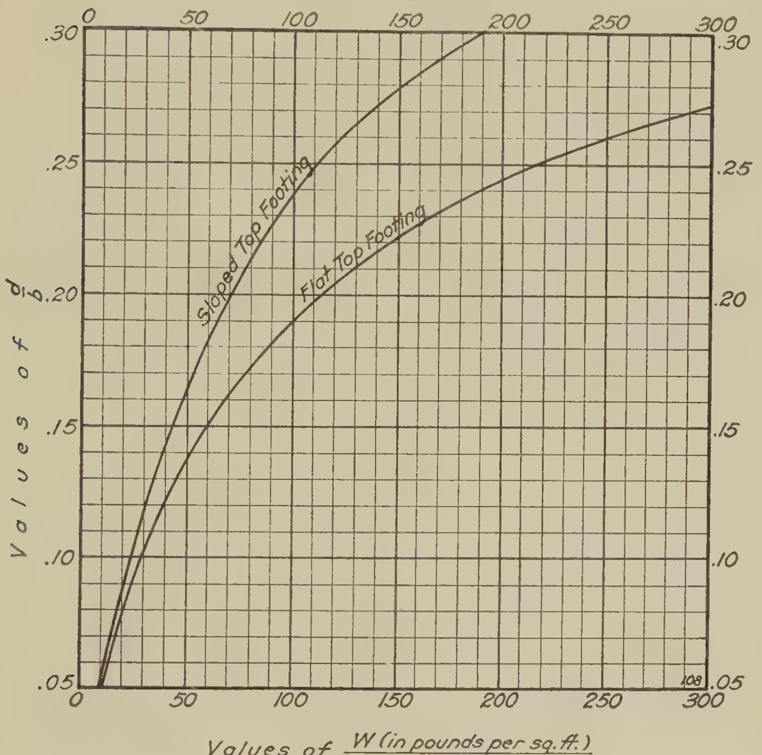
This diagram is based on special anchorage of footing bars where higher shearing stresses (0.03 f'_c) are used in the design. It applies only to square footings, either sloped or flat topped, of the standardized proportions shown in Figs. 16 and 17. The *length* of footing bars must be equal to the overall width of the base of the footing plus twenty bar diameters.

To determine the maximum bar size enter the diagram with the side of the footing, and proceed vertically to the sloping index line marked with the strength of the concrete (2000 to 3750-lb.) and the kind of bar (plain or deformed) to be used. From this intersection pass horizontally to the marginal scales and use a bar not larger than the size at or next below this point on the scale.

INSTRUCTIONS FOR USE OF DIAGRAMS 113, 115, 117, ETC., TO 143

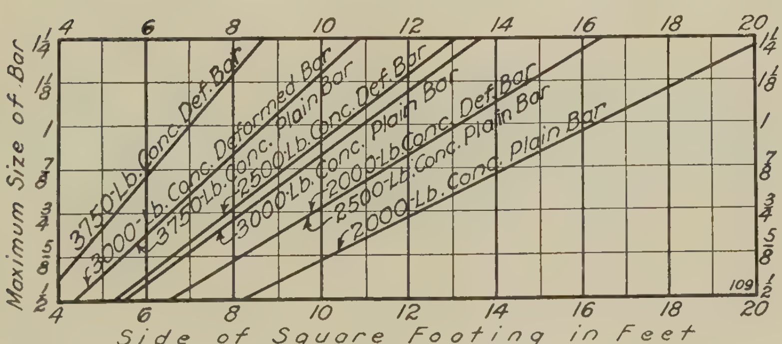
Enter the diagram at the top with the load to be applied at the top of the footing. For small piers this will be the same as the basement story column load. Proceed vertically to an intersection with that one of the upper index lines marked with the strength of the concrete to be used in the design and read off the volume of concrete required for one footing (without pier). Proceed vertically to an intersection with the middle group of index lines and here read off the square feet of formwork required for one footing (exclusive of the pier forms). No forms are included for sloping tops of footings, as the concrete for such footings should be made stiff enough to place without top forms. Continue vertically to an intersection with the lower group of index lines and read off the weight of reinforcing steel required for one footing. No column dowel bars are included and no stirrups or other web reinforcement are necessary. Every bar is bent with a hook at each end.

DIAGRAM 108.—EFFECTIVE DEPTH OF FOOTINGS



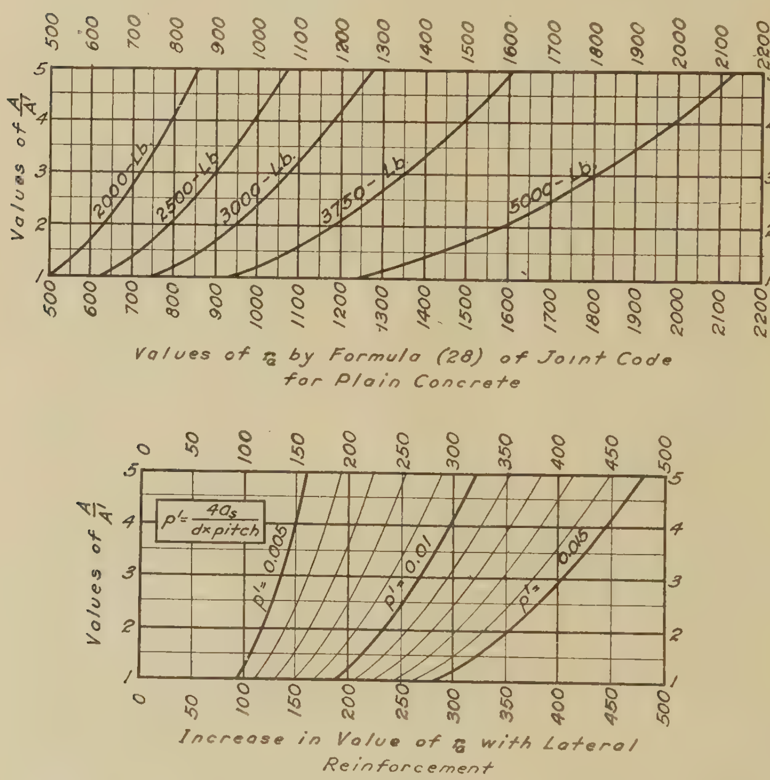
See instructions for use under Fig. 16, page 148.

DIAGRAM 109.—MAXIMUM BAR SIZE IN FOOTINGS



See instructions for use under Fig. 16, page 684.

DIAGRAM 110.—TRANSFER OF COLUMN LOAD TO TOP OF PIER

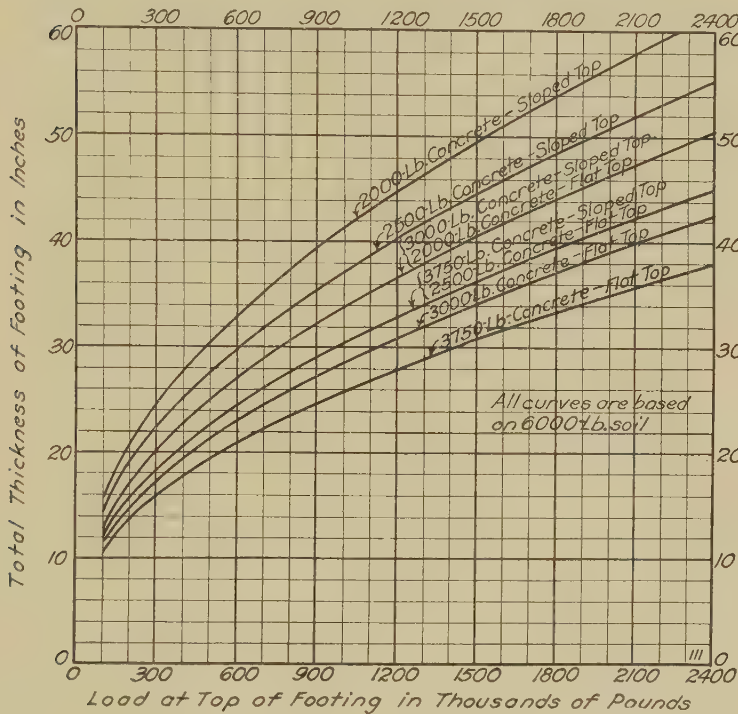


INSTRUCTIONS FOR USE.—In this diagram
A is the area of the top of pier or footing.
A' is the loaded area of the top of pier or footing that is the area of the column section used in design.
r_a is the permissible unit load on the loaded area.

When using the increased value of r_a (on account of spiral reinforcement in the pier) as given in the lower part of the diagram, the value of A must be taken as the area of the top of the pier within the outside diameter of the spiral and this value of A must be used in both the upper and lower portions of the diagram and the two values of r_a added, to get the total permissible unit load where a spiral is used. See instructions under Table 104 for definition of symbols used in formula for p'.

The column vertical bars, or an equal steel area in the form of dowels, must extend at least 24 bar diameters for deformed bars or 30 bar diameters for plain bars above and below the base of column.

DIAGRAM 111.—TOTAL THICKNESS OF FOOTINGS WHEN $v_c = 0.03 f'_c$



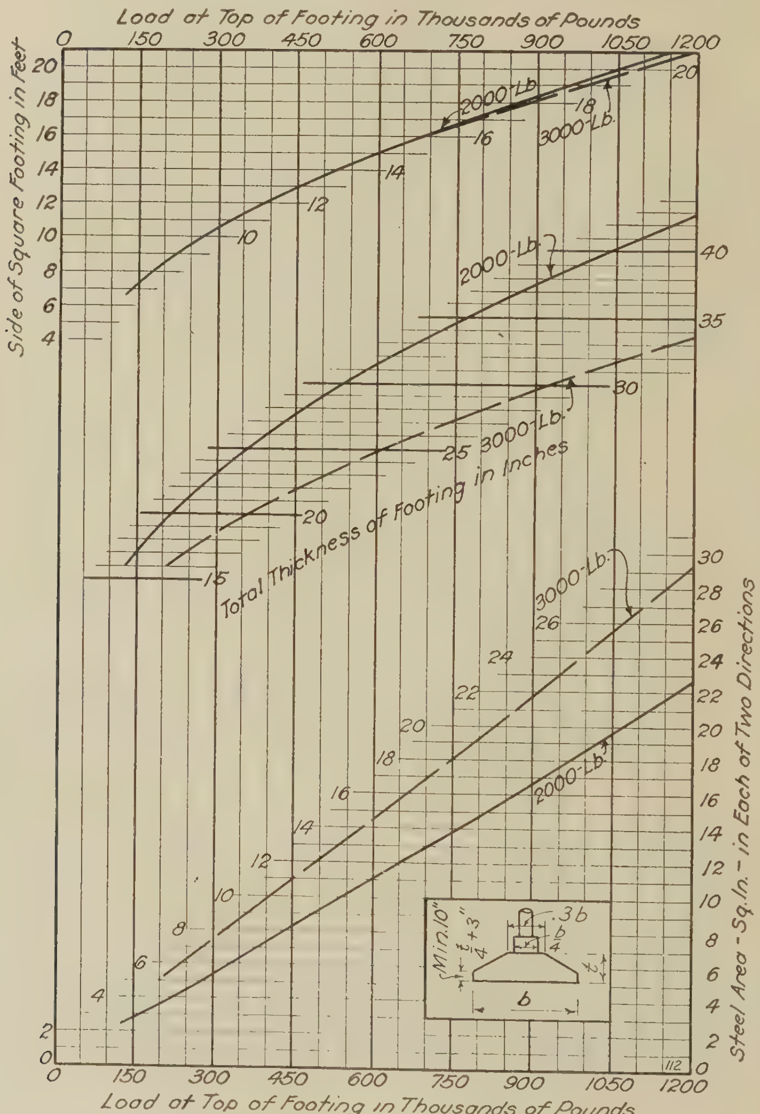
INSTRUCTIONS FOR USE.—This diagram gives an *approximate* indication of the thickness of footing required when the footing bars are anchored as described under Fig. 16. Flat-top footings are based on the standardized proportions shown in Fig. 16 and sloped-top footings on those of Fig. 17. For soil pressures less than 6000-lb. per sq. ft. the total thickness of footings will be slightly less than the values taken from this diagram.

INSTRUCTIONS FOR USE OF DIAGRAMS 112, 114, 116, ETC., TO 142.

Enter this diagram at the top with the load applied at the top of the footing. For small piers this will be the same as the basement column load. Proceed vertically to an intersection with that one of the upper index lines marked with the strength of concrete to be used in the design and read off the dimension in feet of the side of the square base of the footing. Continue vertically to an intersection with the middle group of index lines and here read off the total thickness of the footing in inches. Continue vertically to an intersection with the lower group of index lines and read off the area in sq. in. of the bars required in each of two directions. Use Diagram 109 to determine maximum size of footing bar.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

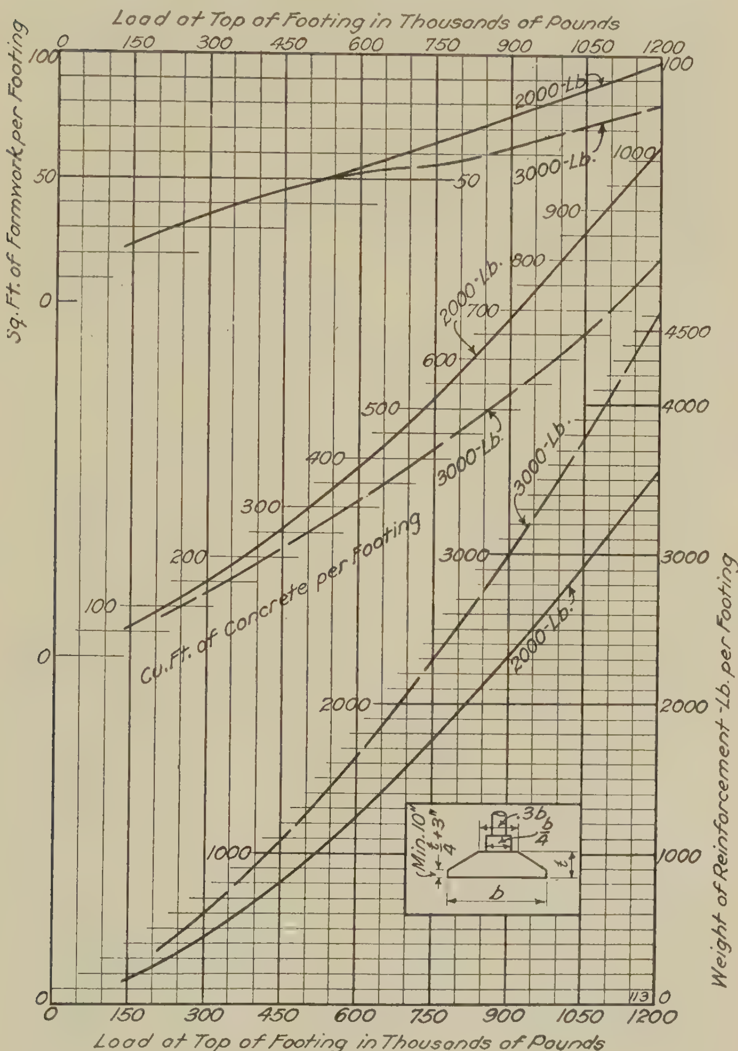
DIAGRAM 112.—3000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

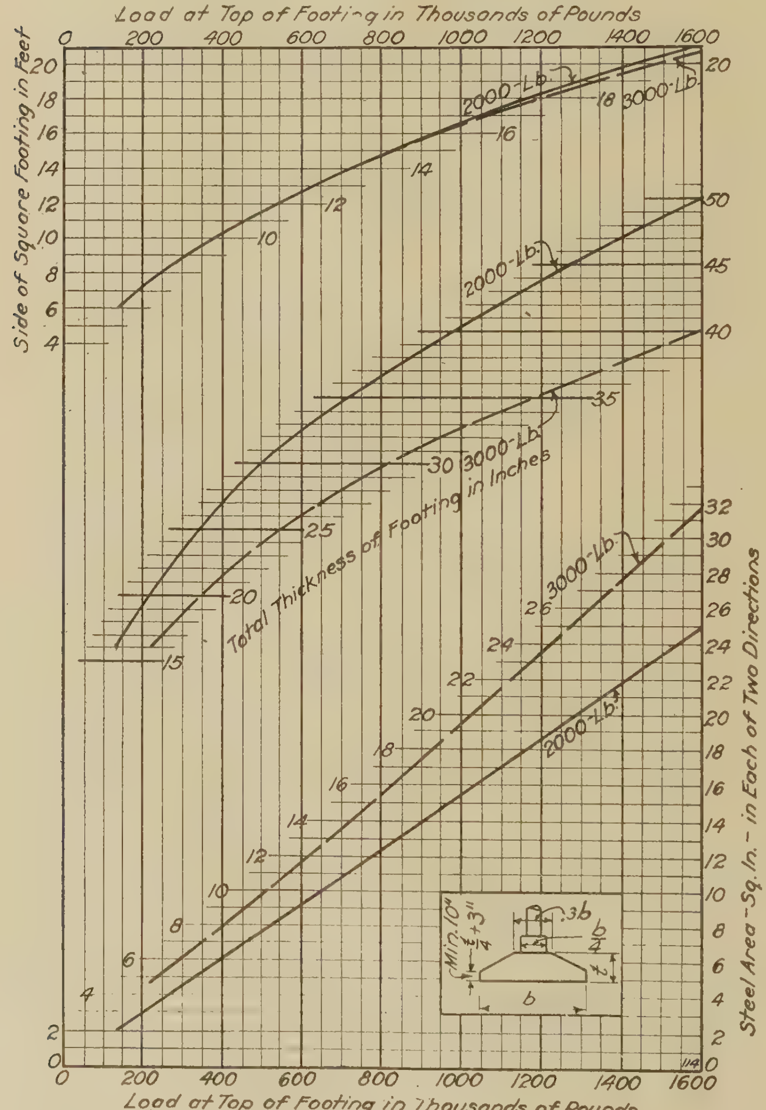
DIAGRAM 113.—3000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

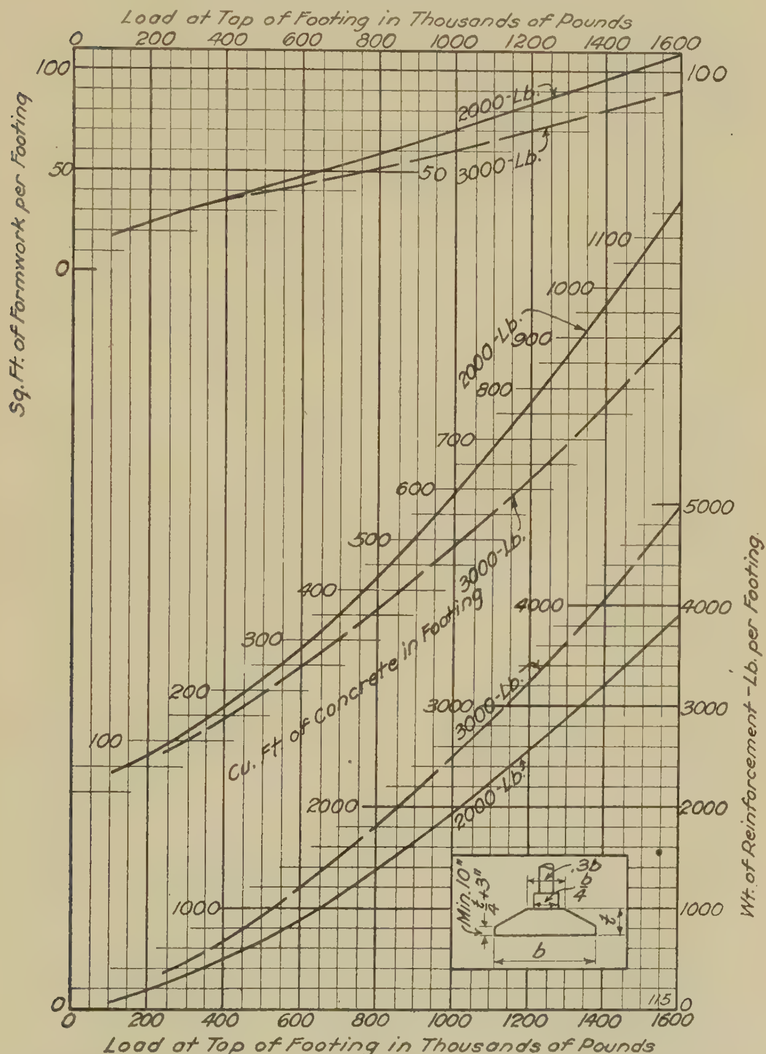
DIAGRAM 114.—4000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

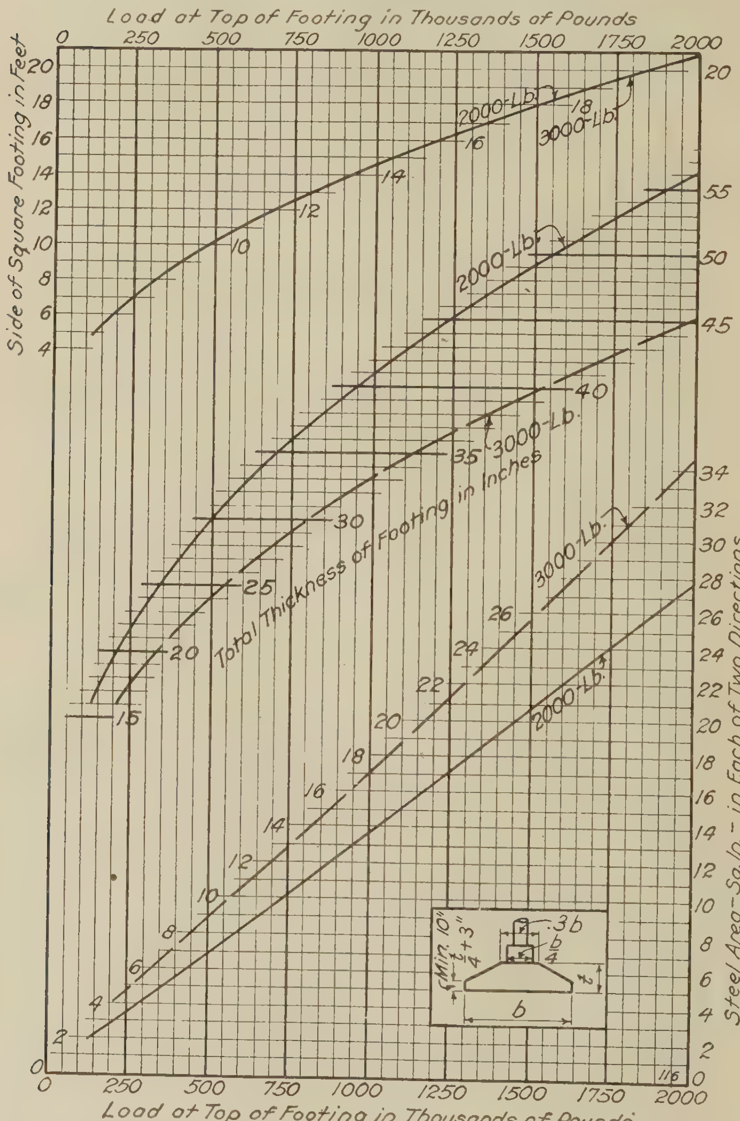
DIAGRAM 115.—4000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

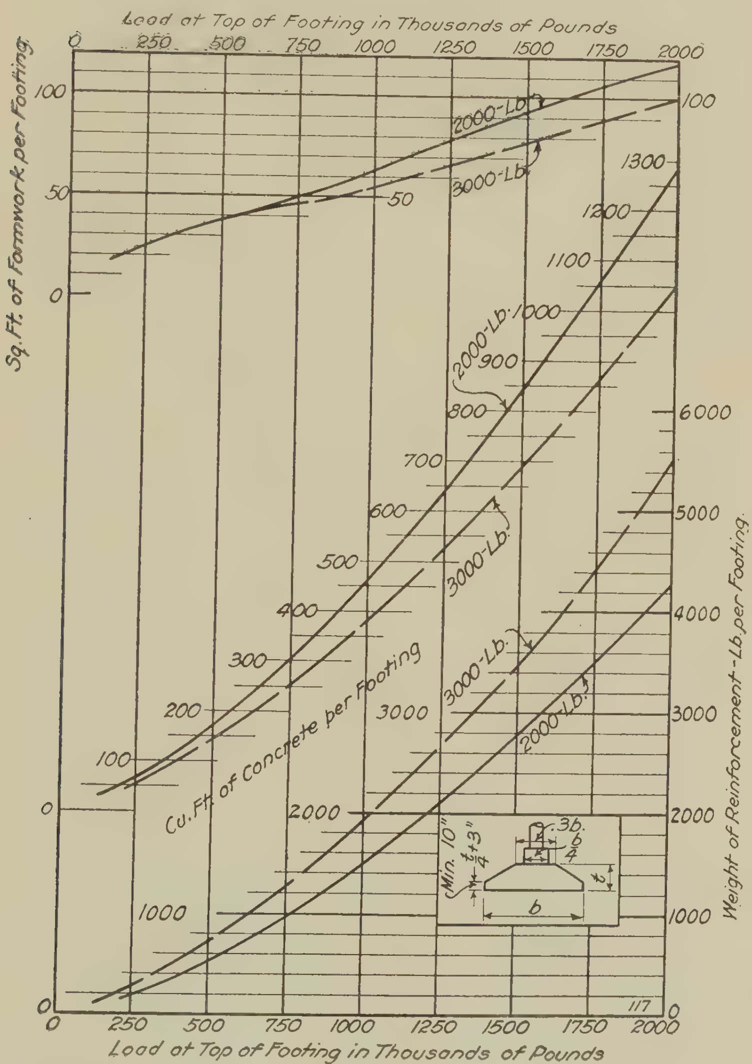
DIAGRAM 116.—5000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

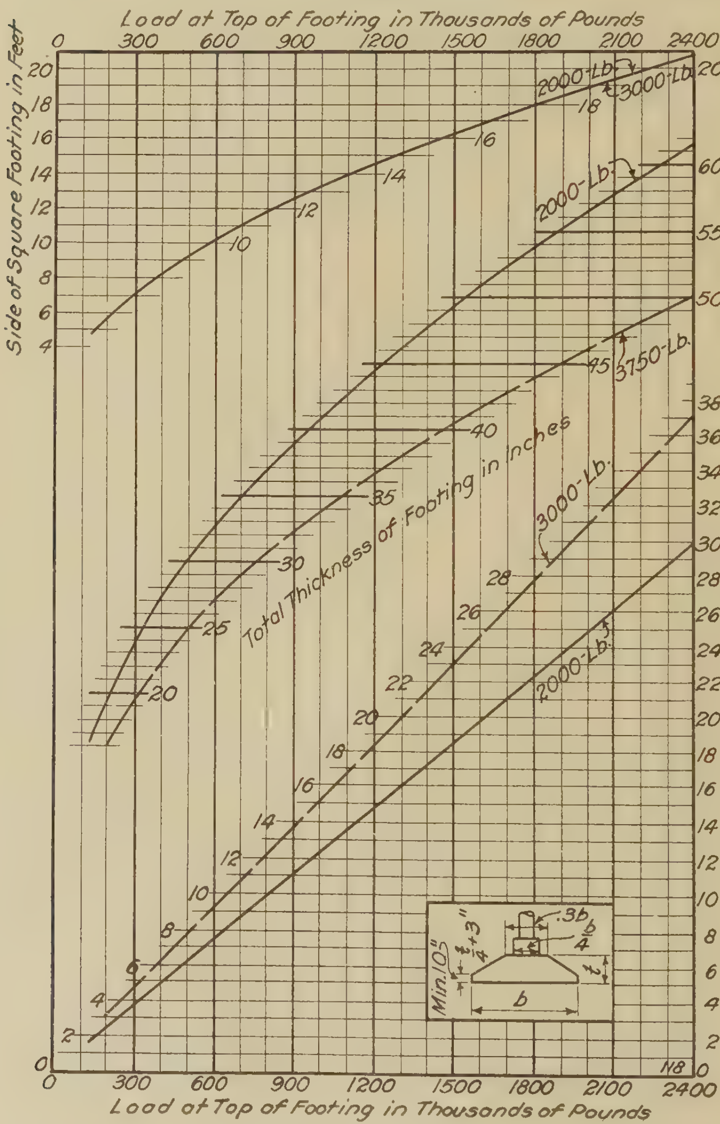
DIAGRAM 117.—5000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

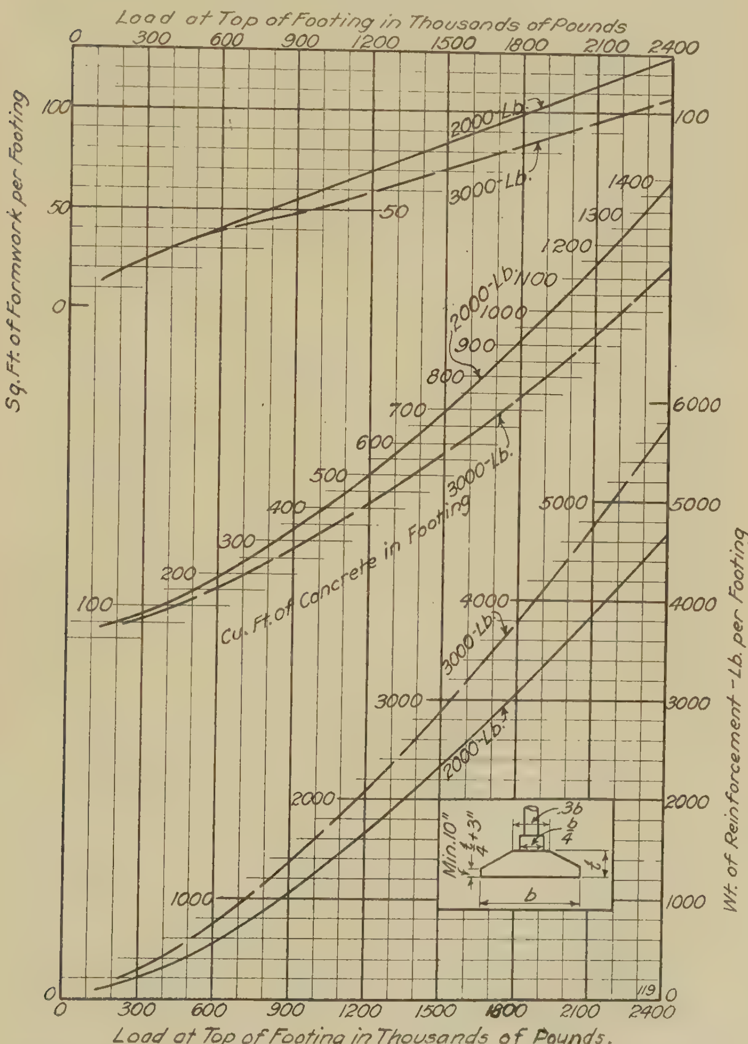
DIAGRAM 118.—6000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

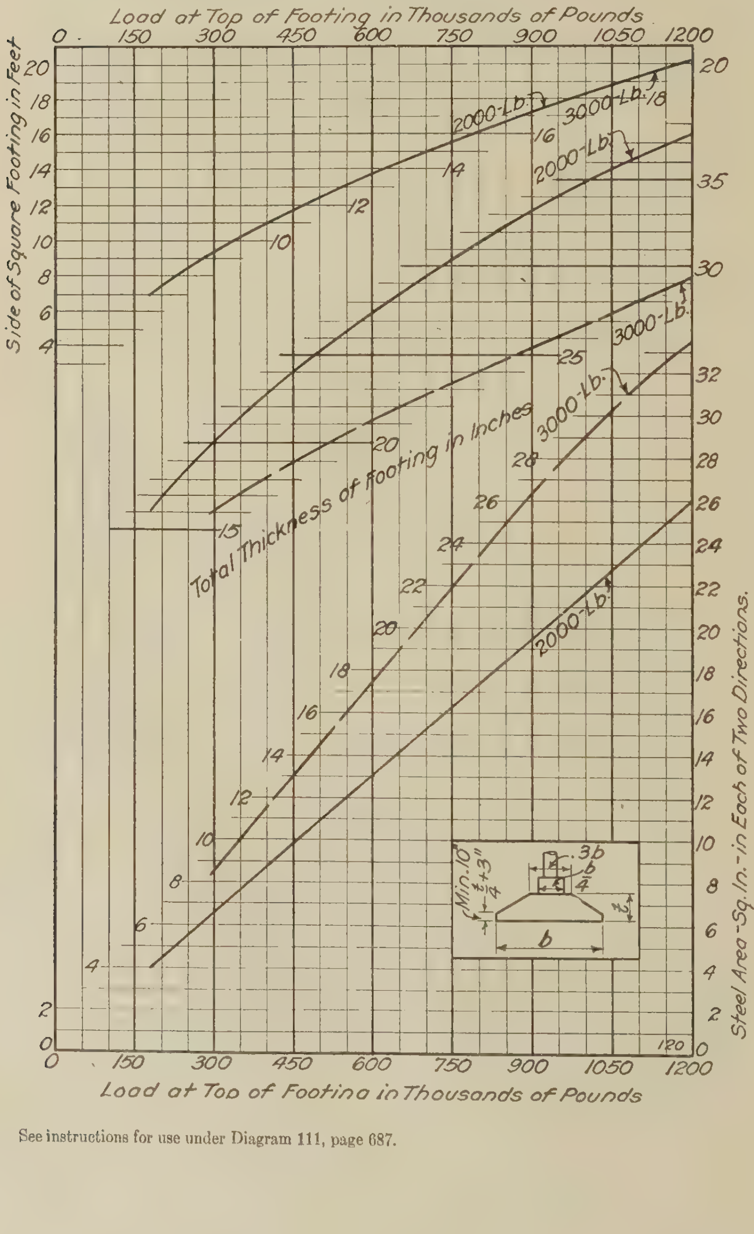
DIAGRAM 119.—6000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

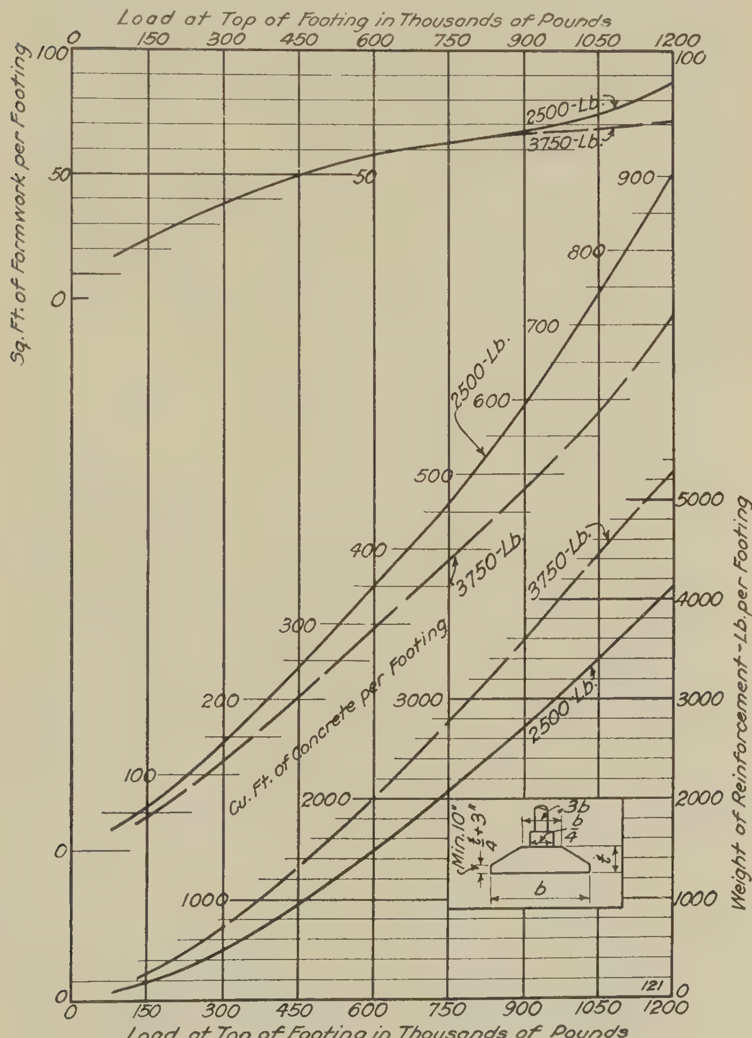
DIAGRAM 120.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

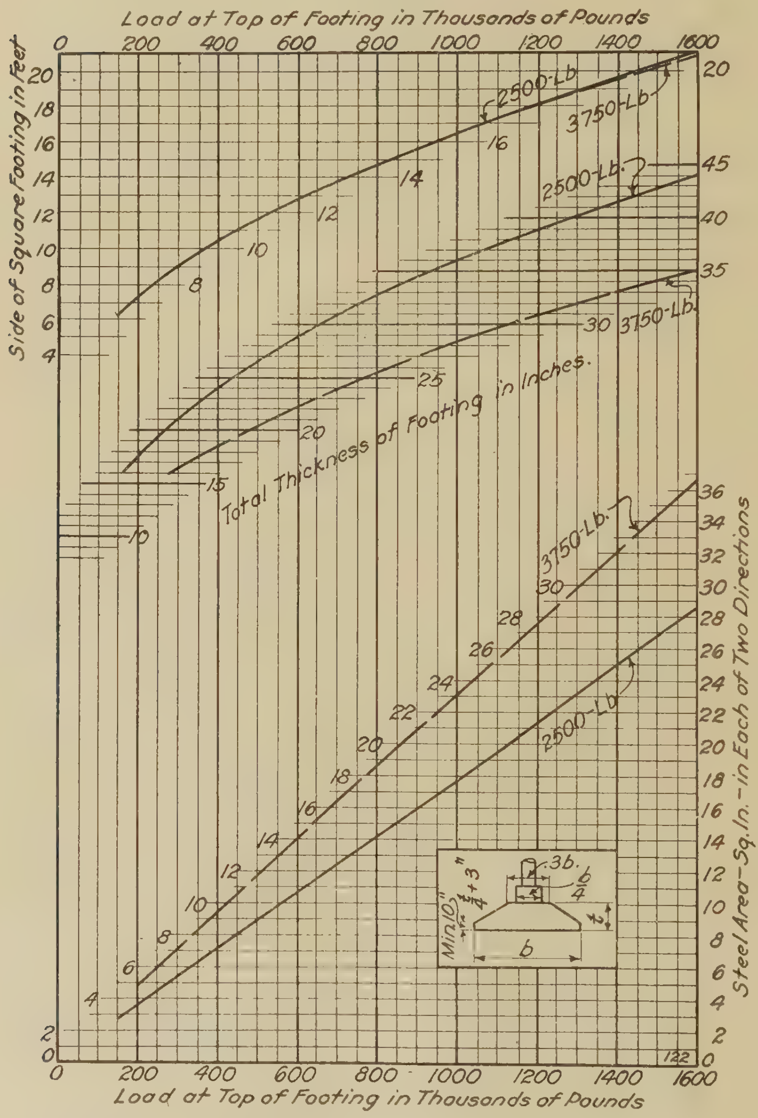
DIAGRAM 121.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

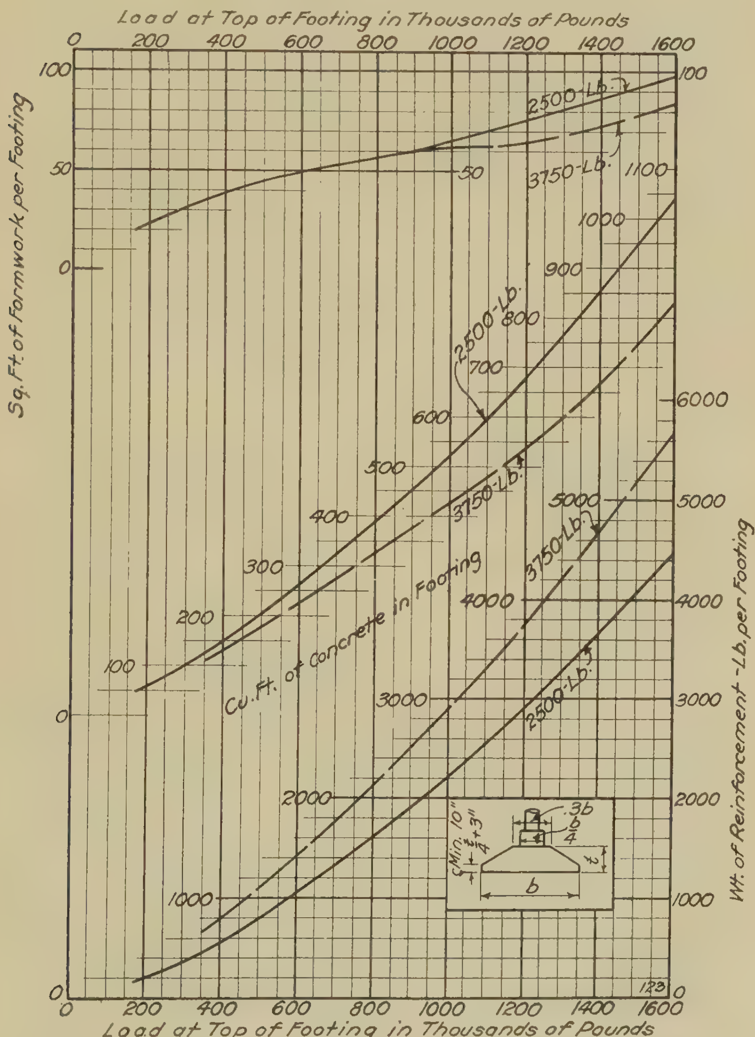
DIAGRAM 122.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

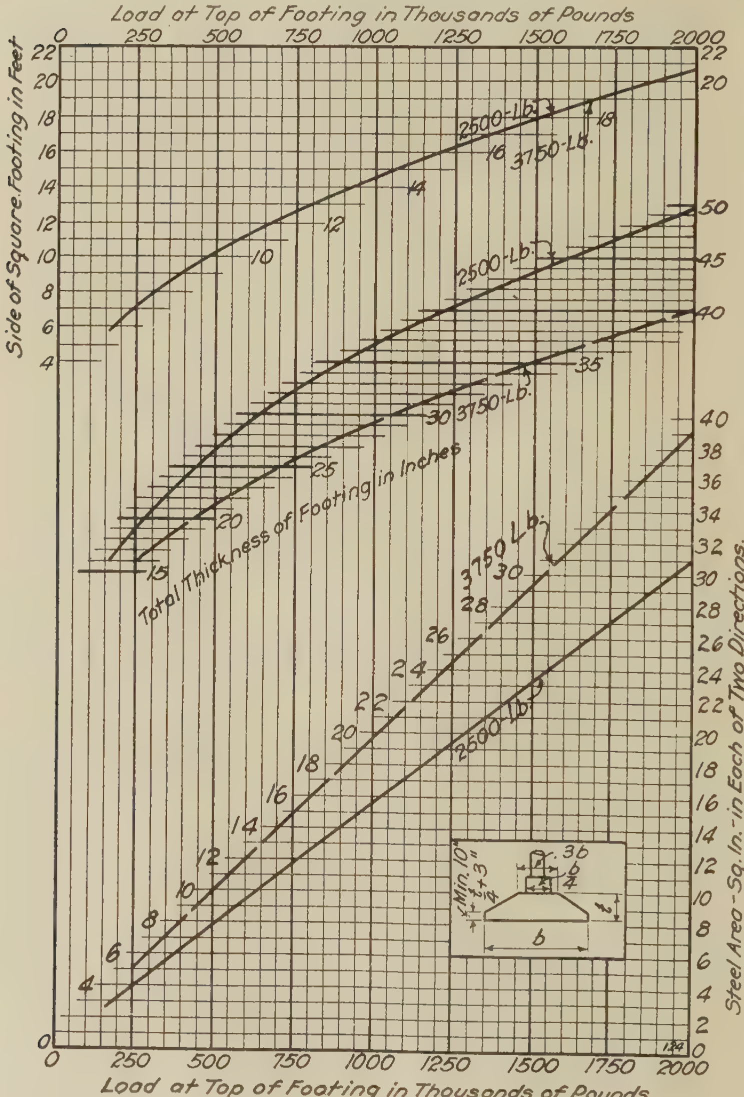
DIAGRAM 123.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

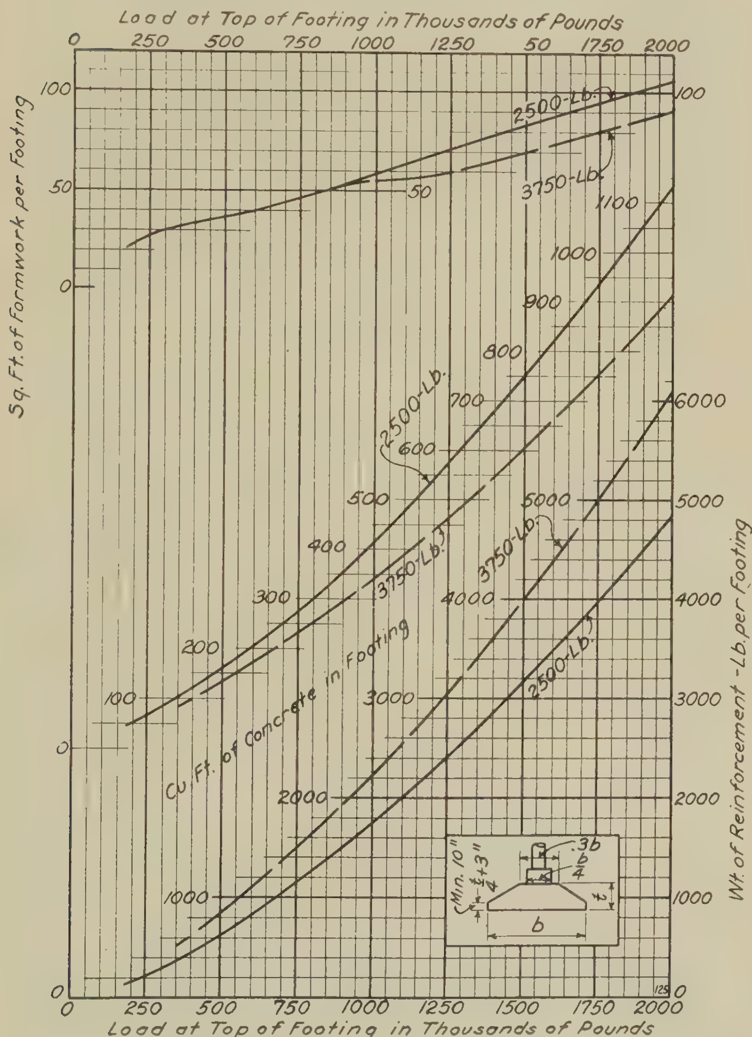
DIAGRAM 124.—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

Quantities for Sloped-Top Footings with $v_c = 0.03f'_c$

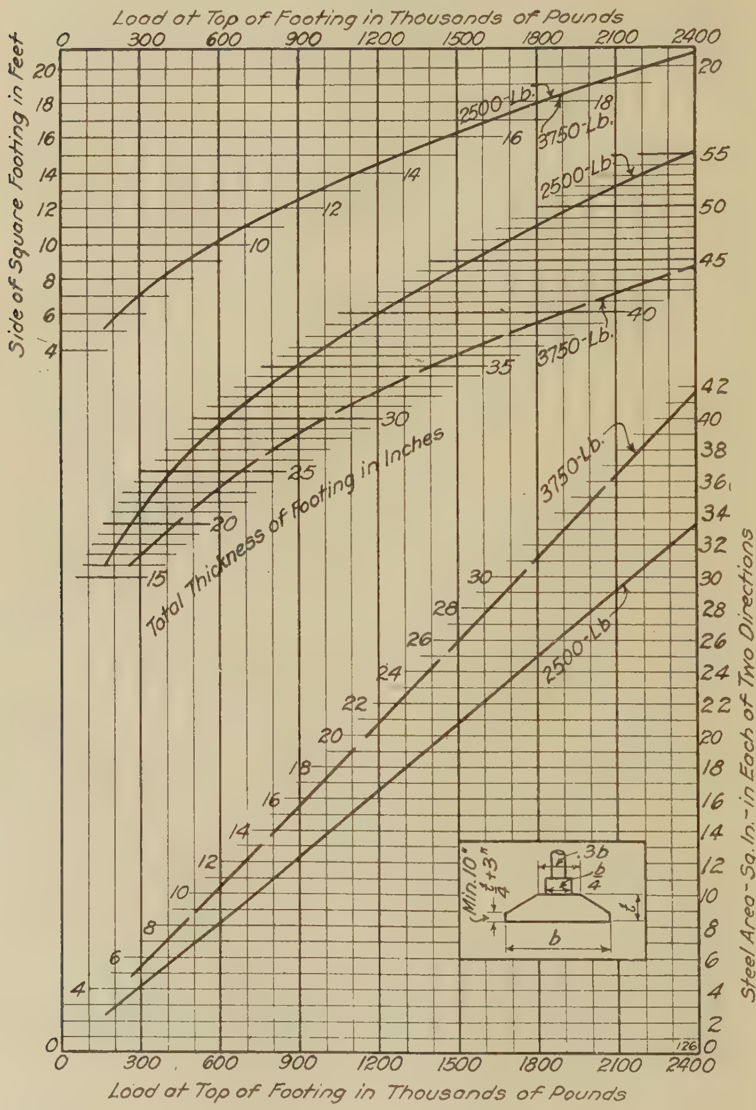
Diagram 125.—5000-lb. Soil—2500-lb. and 3750-lb. Concrete



See instructions for use under Fig. 16, page 684.

DESIGN OF SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

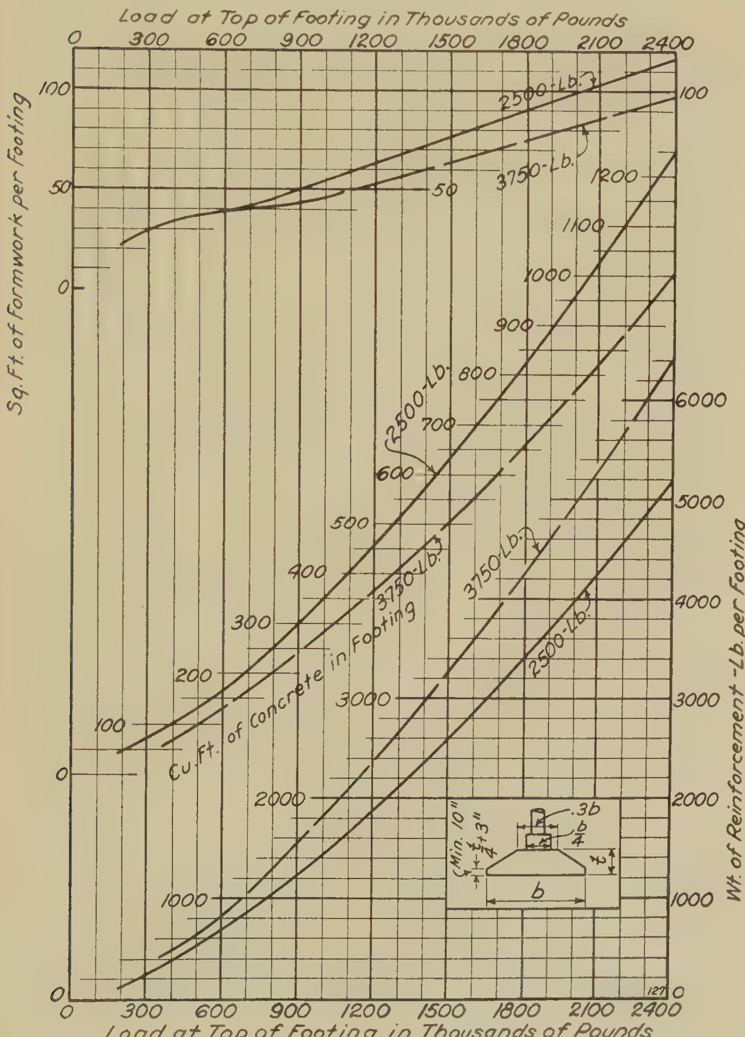
DIAGRAM 126.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR SLOPED-TOP FOOTINGS WITH $v_c = 0.03f'_c$

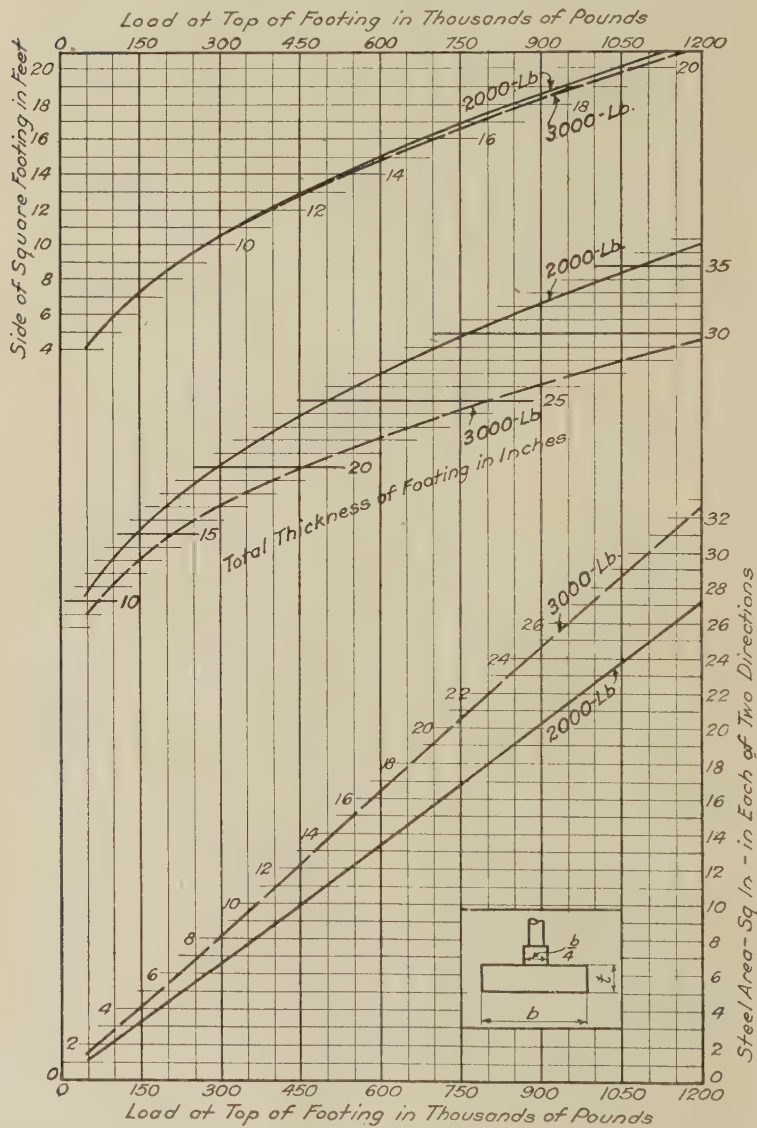
DIAGRAM 127.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

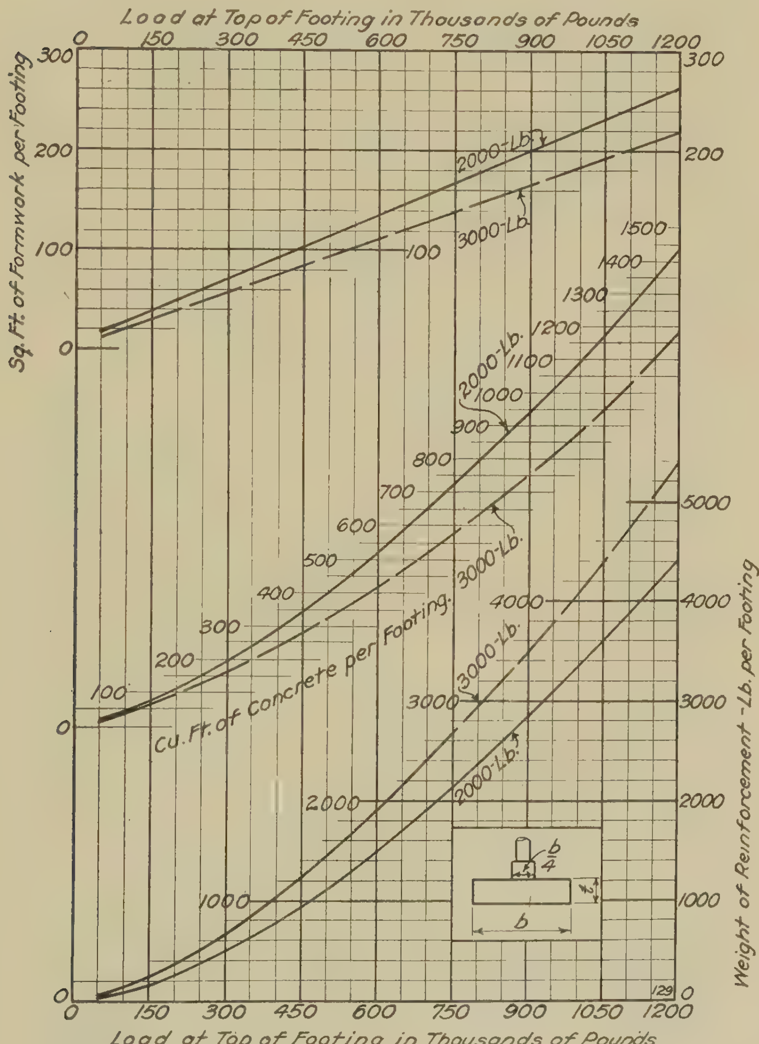
DIAGRAM 128.—3000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

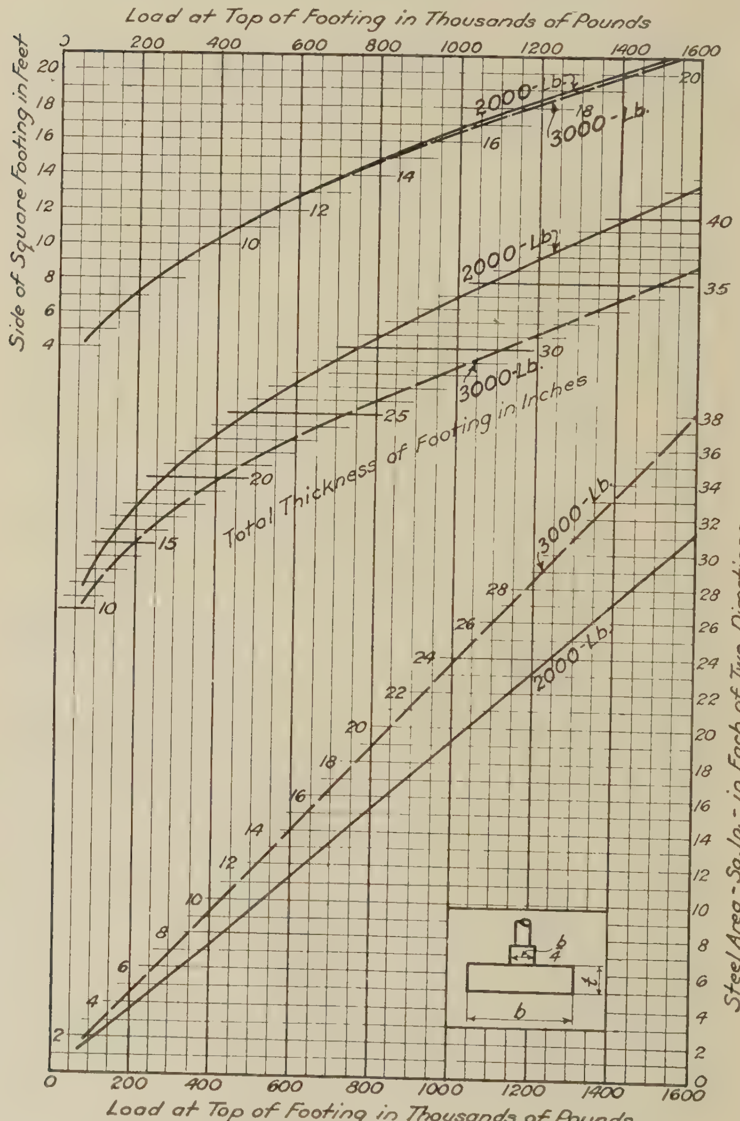
DIAGRAM 129.—3000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03\,f'_c$

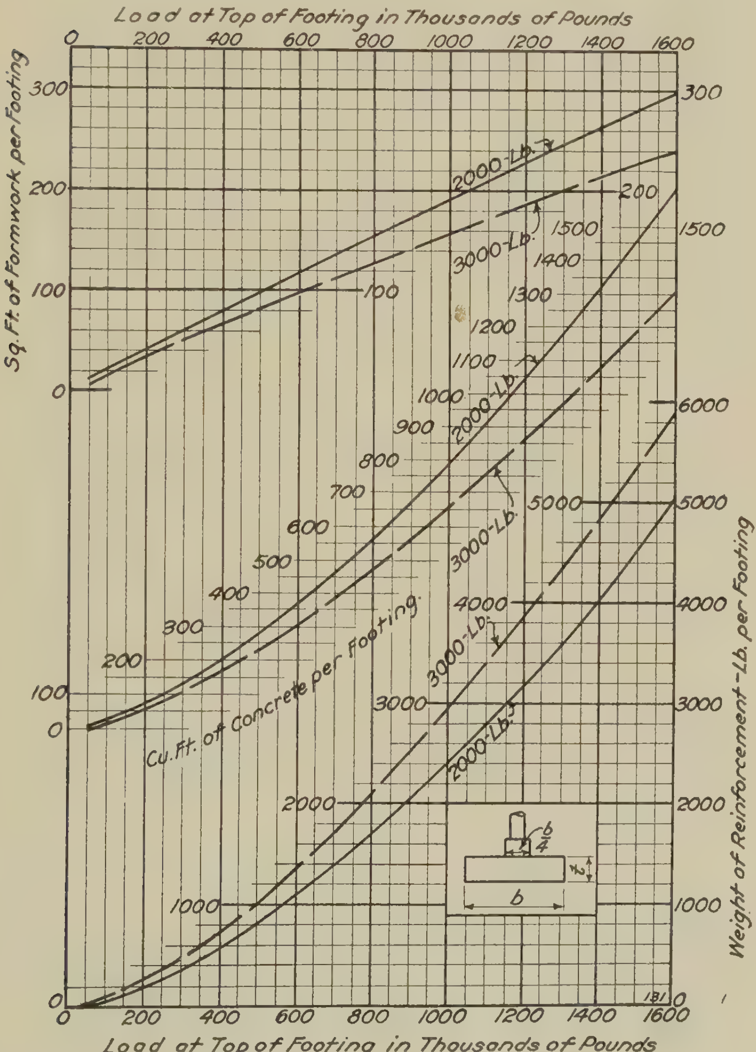
DIAGRAM 130.—4000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

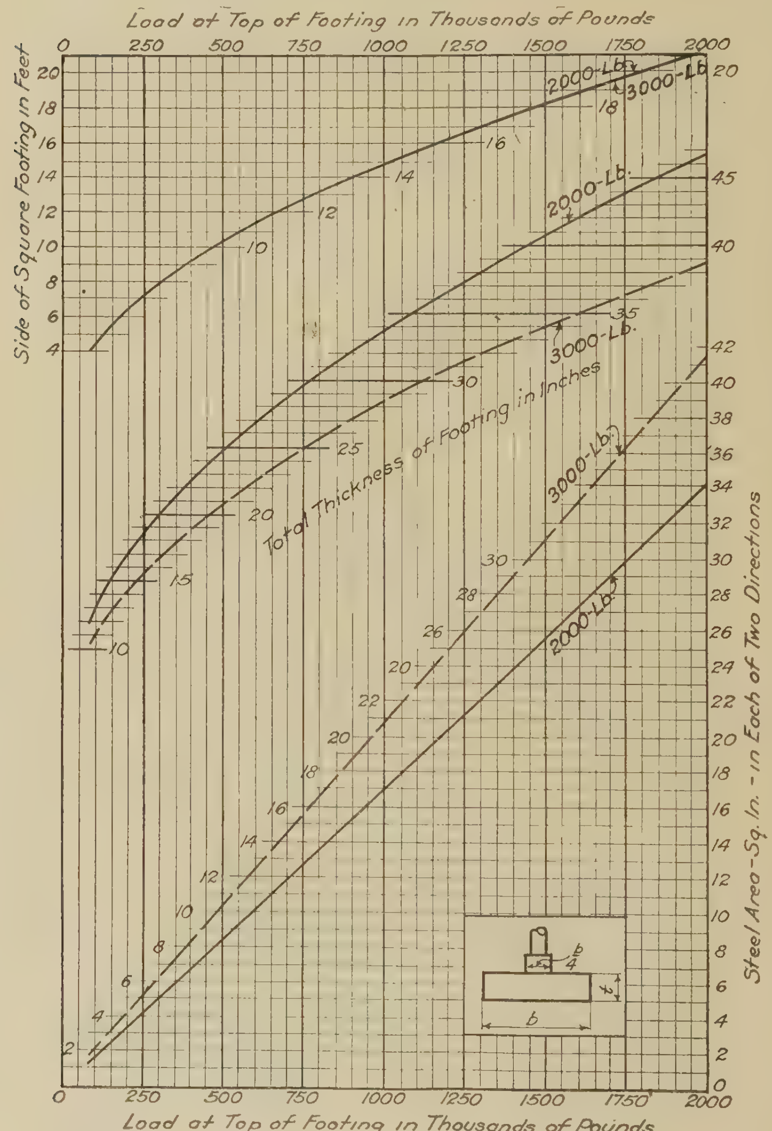
DIAGRAM 131.—4000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f'_c$

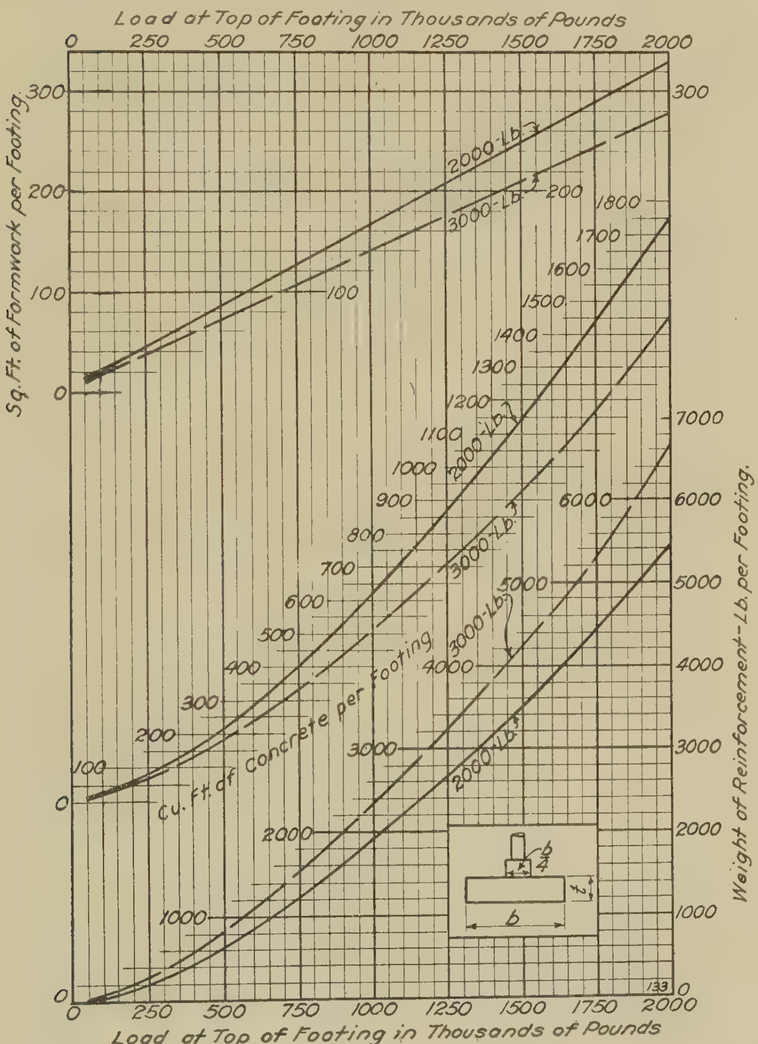
DIAGRAM 132.—5000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f_c'$

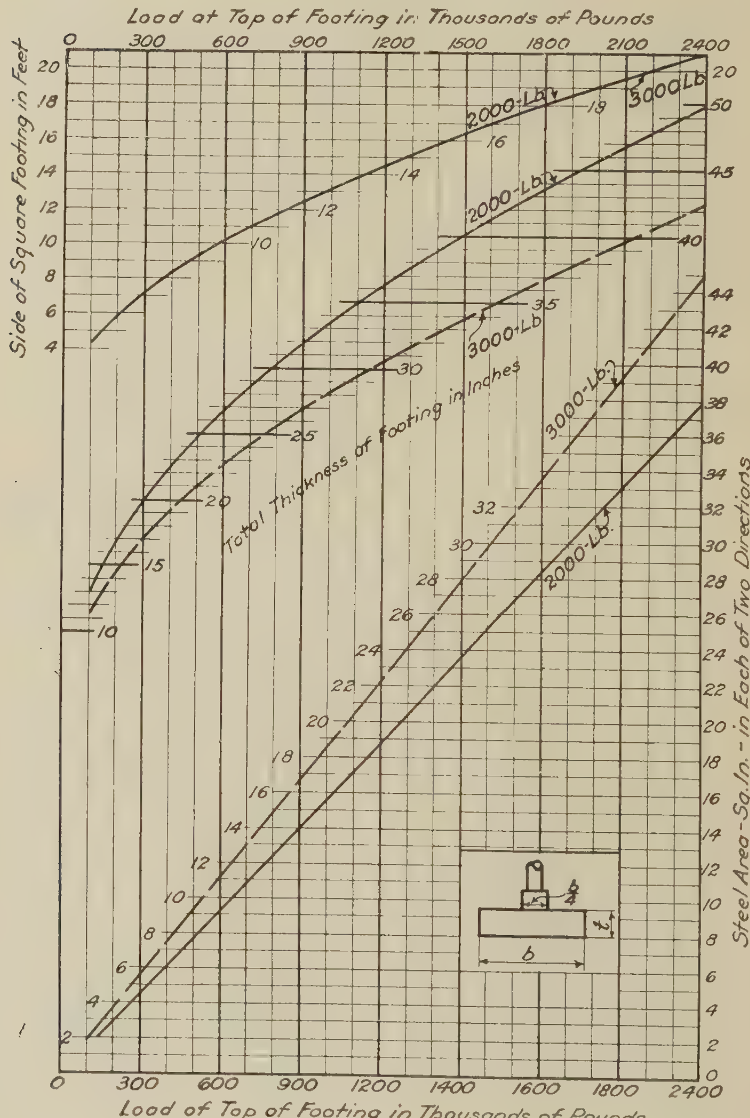
DIAGRAM 133.—5000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03 f_c'$

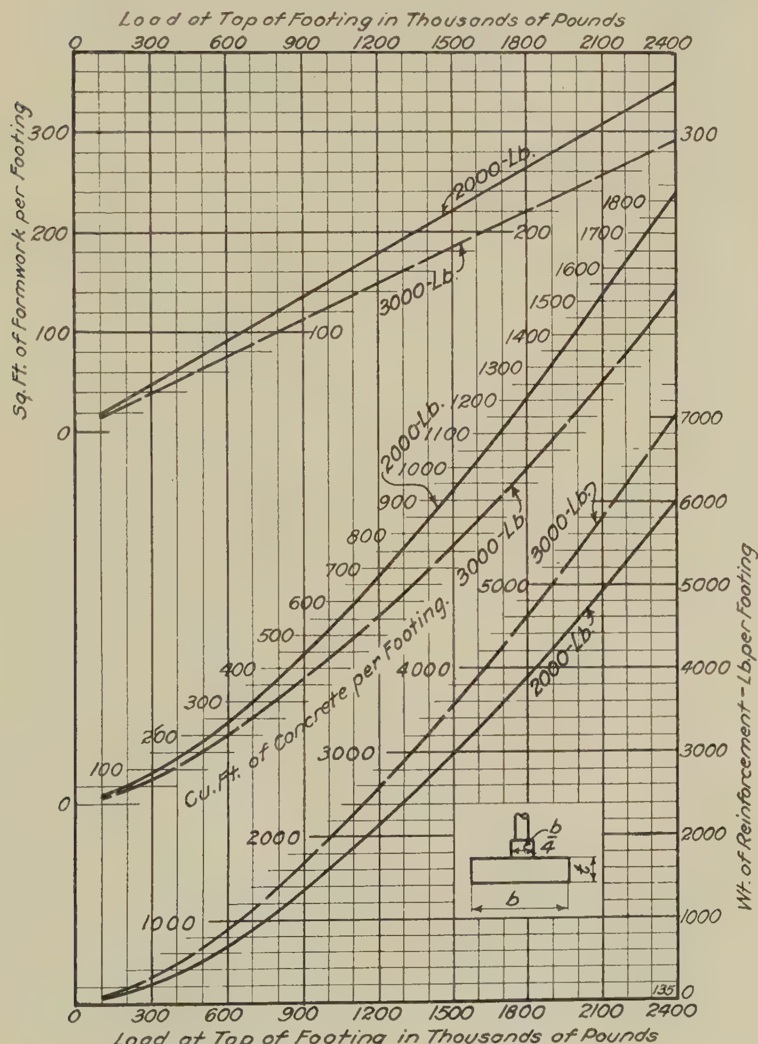
DIAGRAM 134.—6000-LB. SOIL — 2000-LB. AND 3000-LB. CONCRETE



See iustructions for uses under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

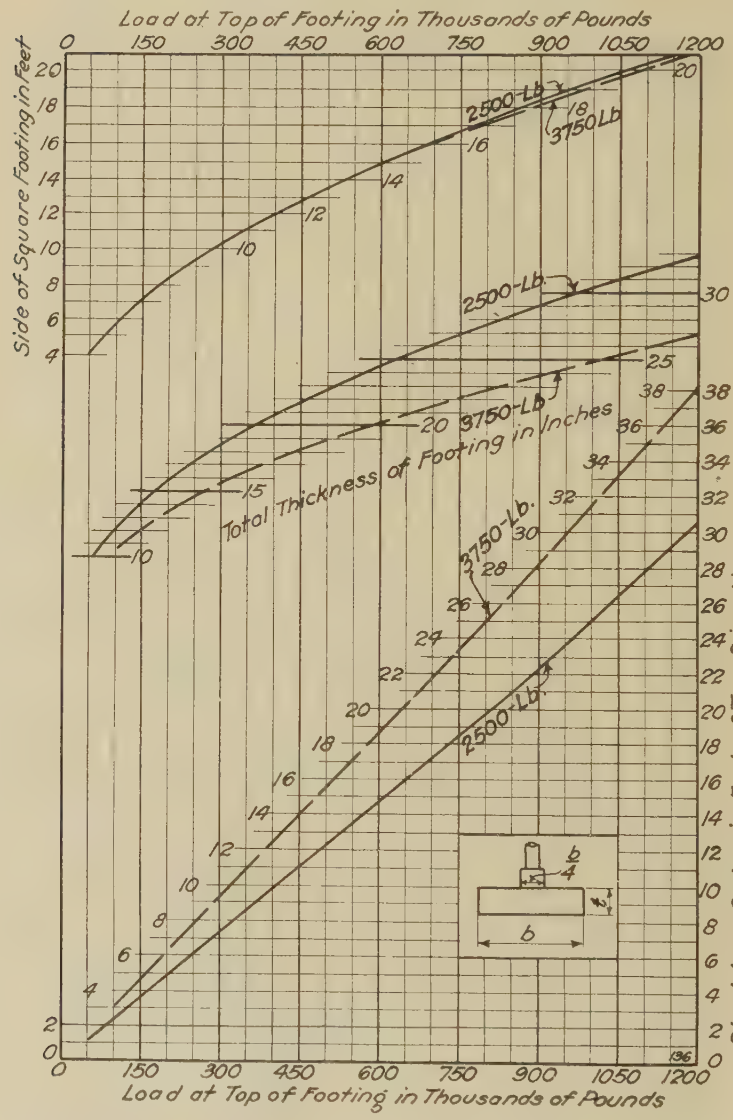
DIAGRAM 135.—6000-LB. SOIL—2000-LB. AND 3000-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

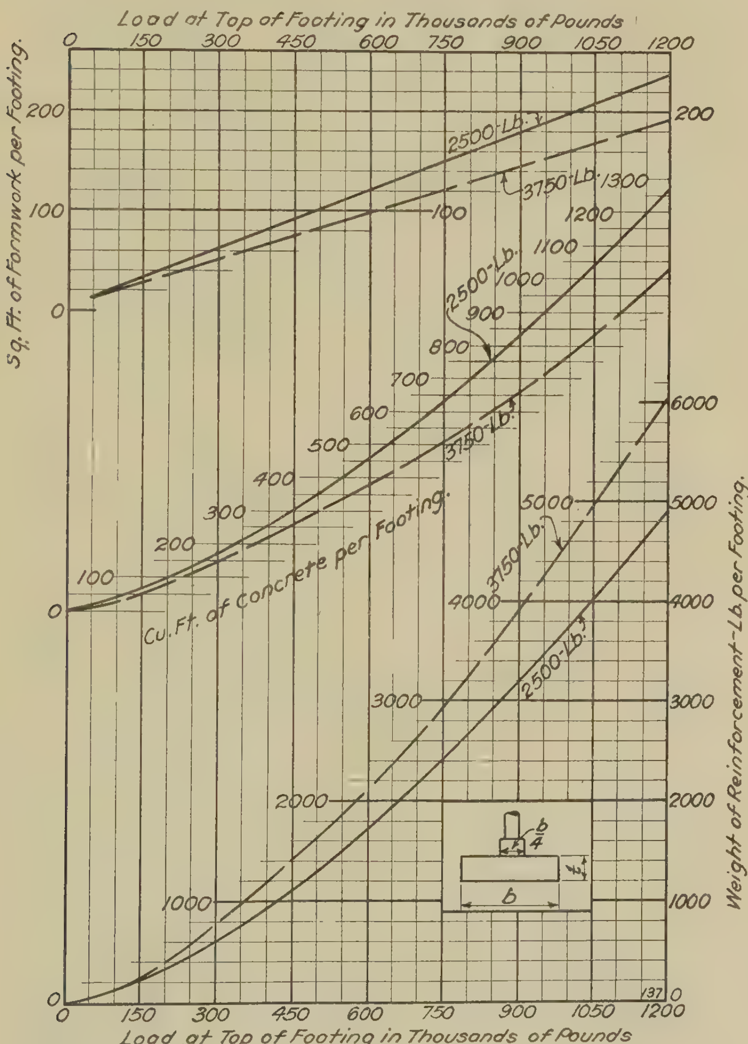
DIAGRAM 136.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

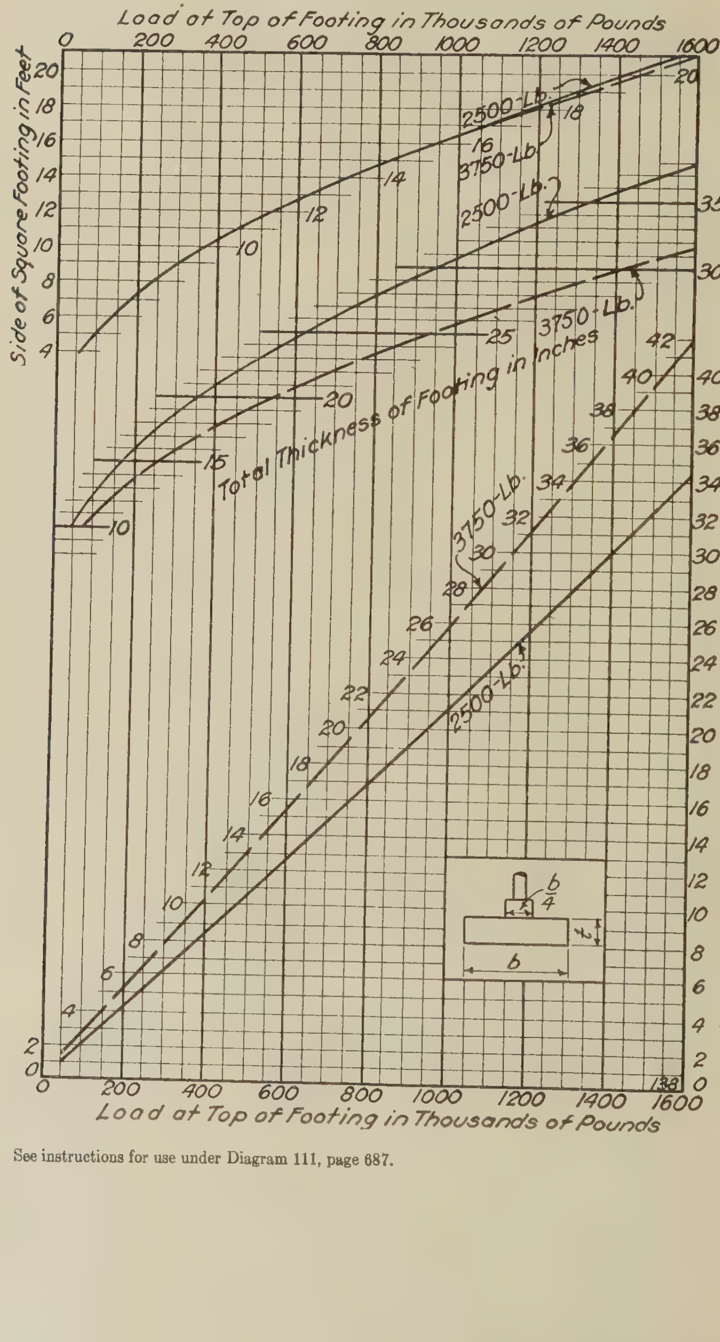
DIAGRAM 137.—3000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

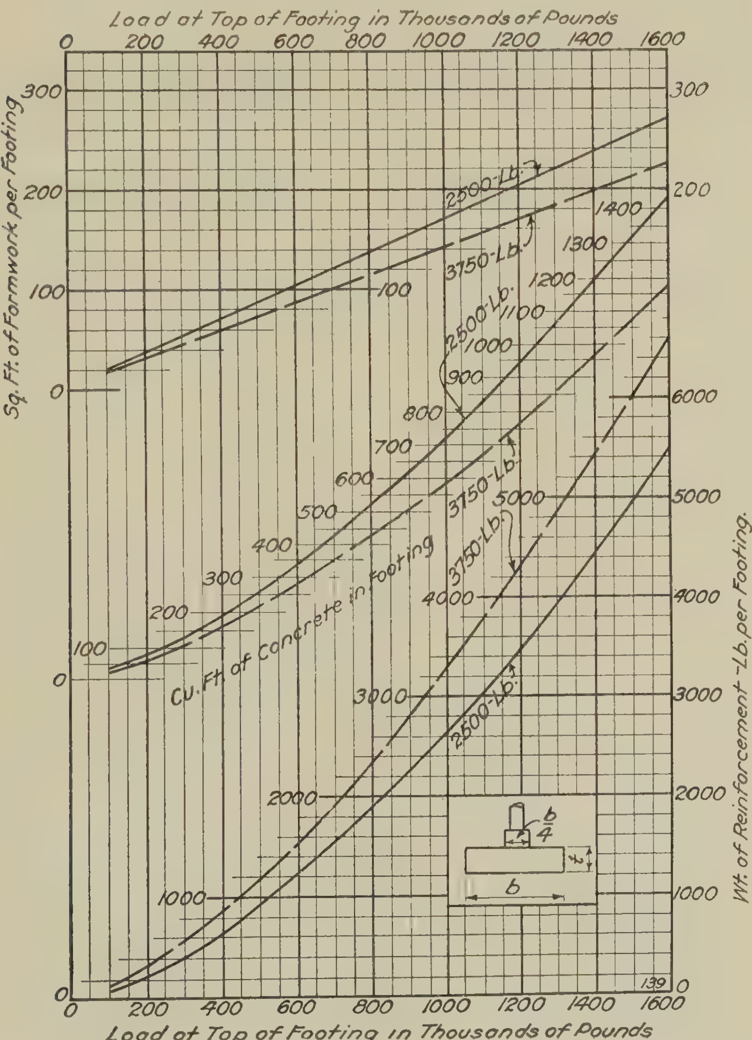
DIAGRAM 138.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

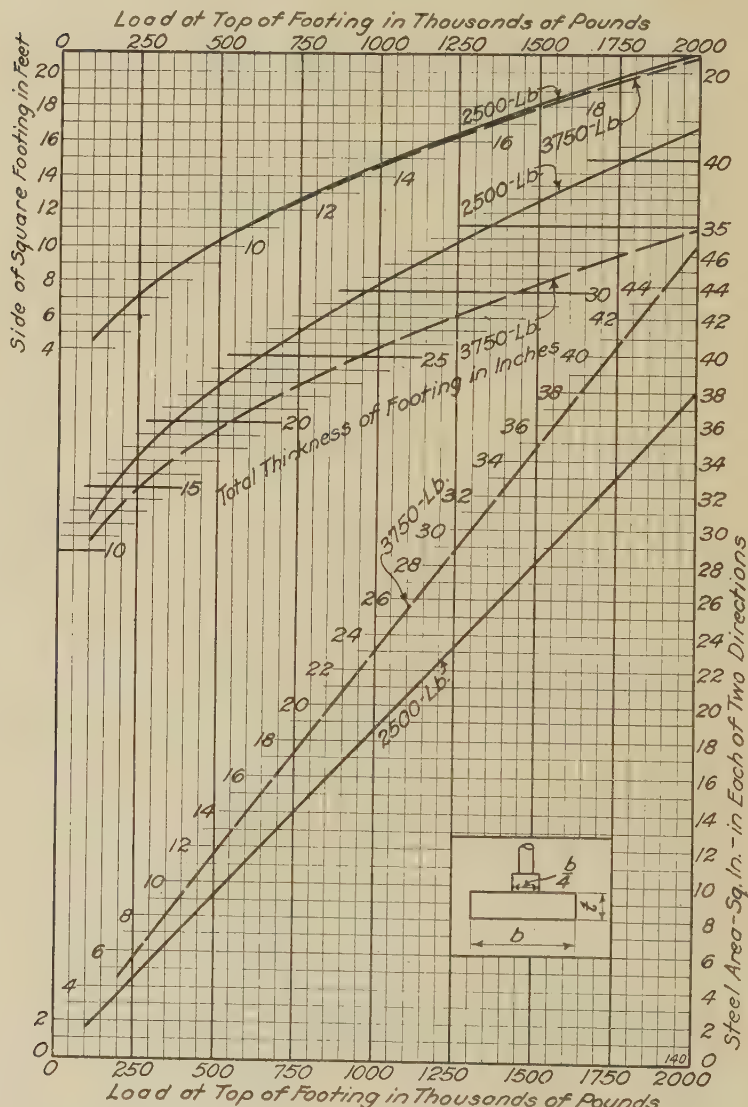
DIAGRAM 139.—4000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

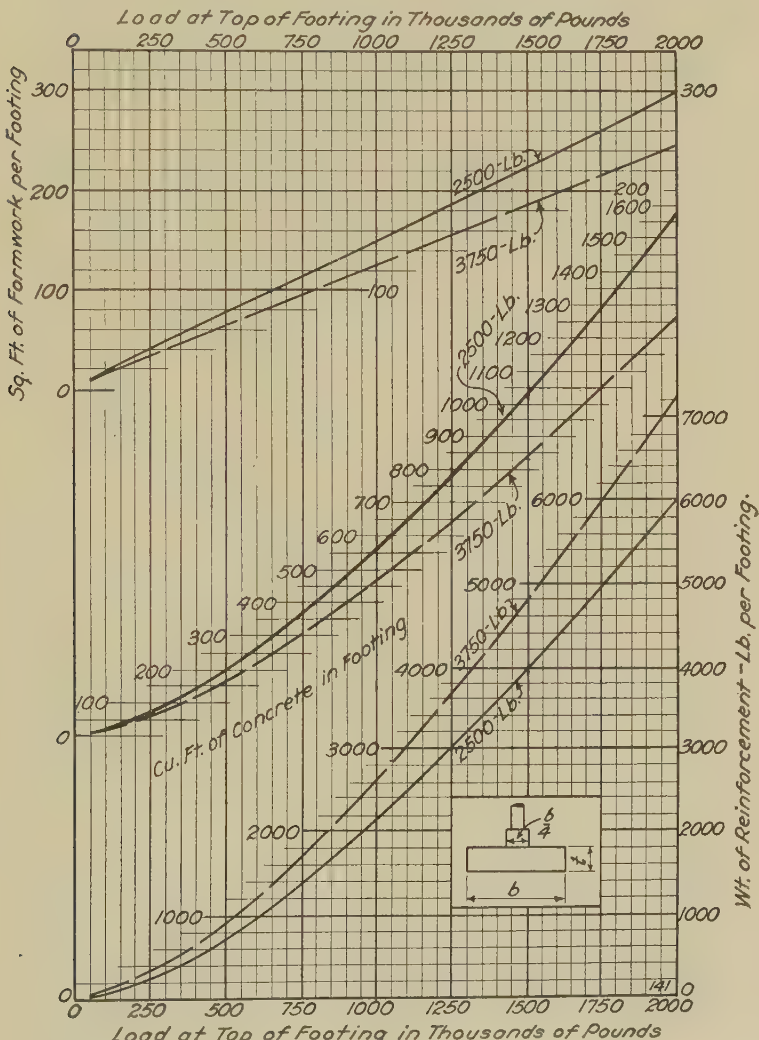
DIAGRAM 140—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

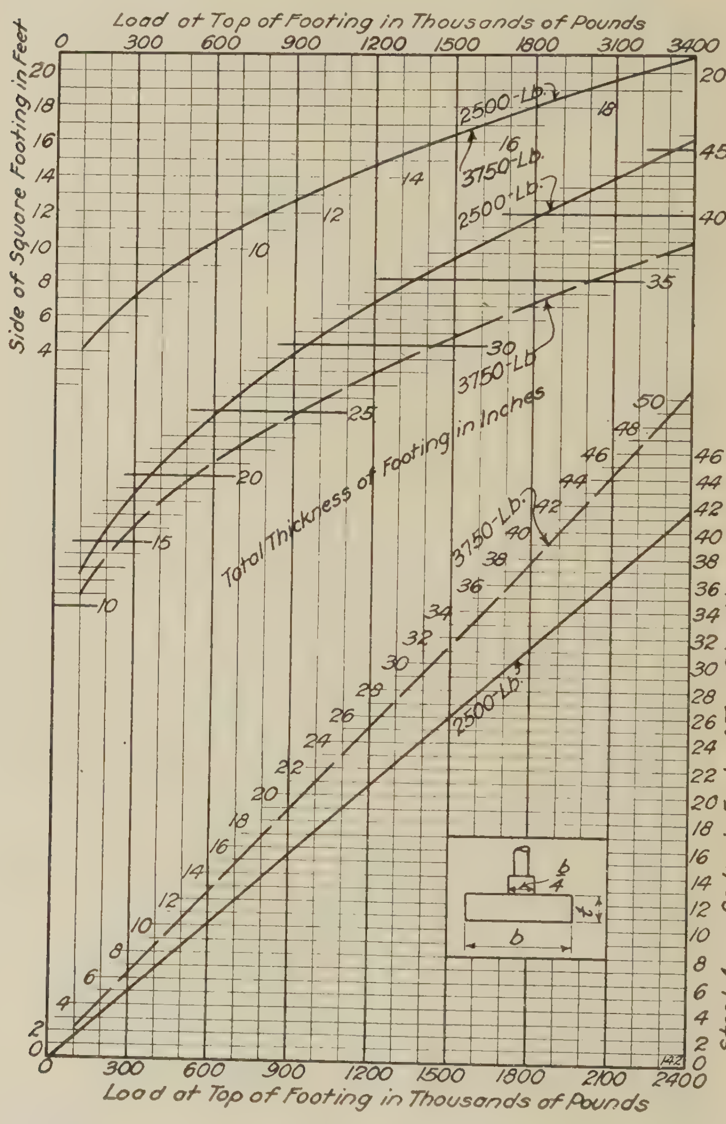
DIAGRAM 141.—5000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

DESIGN OF FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

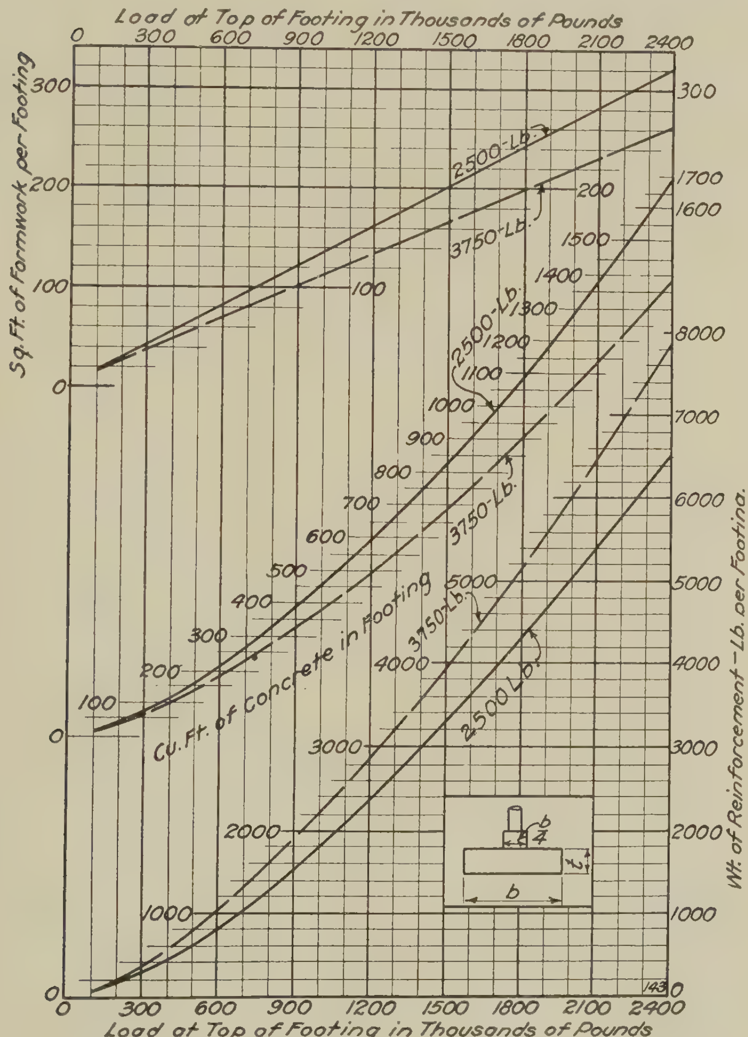
DIAGRAM 142.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Diagram 111, page 687.

QUANTITIES FOR FLAT-TOP FOOTINGS WITH $v_c = 0.03f'_c$

DIAGRAM 143.—6000-LB. SOIL—2500-LB. AND 3750-LB. CONCRETE



See instructions for use under Fig. 16, page 684.

PART TWO—COST DATA.

The cost of reinforced-concrete building construction designed in accordance with the 1928 Joint Standard Building Code will vary between fairly wide limits in different parts of the country. Where some cities pay $2.50 or more per cubic yard for concrete aggregate it may sometimes be secured on country jobs for as low as 60 cents. The price of cement varies considerably and the labor cost is subject to great variations. Many other factors, including plant, size of job, number of stories, contracting ability, weather and working conditions, and labor rules contribute to the variation in building cost. I have therefore given in Tables 145 to 156 the *quantities* of concrete, reinforcing steel, formwork, excavation, etc., from which the engineer may compute for himself with little trouble the local price differential for the conditions on which he must operate. In Table 144 and Fig. 18 unit prices have been used, as given later, and the cost comparison given in terms of money.

I have made complete designs of a typical interior panel, from foundation to roof, for 96 buildings covering the following range:

2 Types of Construction	:	Flat Slab and Beam-and-Girder.
2 Concrete Strengths	:	2,000 lb. and 3,000 lb.
2 Panel Sizes	:	18 ft. by 18 ft. and 22 ft. 6 in. by 22 ft. 6 in.
2 Live Loads	:	100 lb. and 300 lb. per sq. ft.
2 Soil Pressures	:	3,000 lb. and 6,000 lb. per sq. ft.
3 Building Heights	:	6, 9 and 12 stories.

In selecting the values for the variables, two choices have been made representing a fairly low and a fairly high value. Thus an 18-ft. square panel is fairly small while a 22-ft. 6-in. square panel is fairly large. A 100-lb. live load is small, a 300-lb. live load is large. A six-story building is quite low and a twelve-story building fairly tall, as current work goes.

The quantities reported are for the structural framework of the building, including roof and floor slabs, but not including exterior walls, brick masonry, architectural finish or equipment of any kind. The design was carried out in general conformity with the Joint Code, using the tables and diagrams given in this paper. In designing, the size of column has been maintained the same for all parallel designs between the usual 2,000-lb. concrete structure and the structure using stronger concrete. The usual design, however, involves 3,000-lb. concrete in the lower story columns while the higher-concrete-strength design uses 2,000-lb. concrete in the top stories increasing to 5,000-lb. concrete in the lower stories. The roof, floor slabs and the footings are designed for one concrete strength in any one building—2,000 lb. or 3,000 lb. as the tables indicate. It would be possible to manipulate such a comparison to a considerable extent, but my intention, carried out with much care and thought, has been to make designs in the 2,000-lb. and 3,000-lb. classes, which would represent equally good practice in both cases and which would show only that saving in materials which naturally results from the use of richer concrete. One of the chief savings is the reduction in dead weight of the structure itself which shows up mainly in the column and footing quantities. The saving resulting from the use of richer concrete may be realized in either one of two ways: (1) as in done in this paper, the first cost of the building may be reduced, or (2) the size of columns may be greatly reduced resulting in a valuable increase in the useable floor areas.

In Tables 145 to 152 inclusive, quantities are stated for the elements of which buildings are composed—roof, floors, columns, footings—and numerous buildings may be worked out from these tables which are not included in Tables 80 to 156 inclusive, where the total quantities for one panel from footing to roof for 96 buildings have been assembled. The concrete quan-

tities do not include a basement floor on the ground. Formwork has been given in square feet of concrete surface in contact with the forms, except that steel column forms are given in units for one story. Reinforcing steel is separated, between bars (both straight and bent) and spiral (including ties and stirrups in the spiral item). Excavation allows 4 inches on all sides of the footing and excludes 6 inches of the combined depth of footing and pedestal. This is pit excavation only. In four cases where 3,000-lb. soil would not support the column load within the panel area, concrete piles are indicated, computed for a load of 30 tons each. Items containing piles suffer a considerable increase in cost, but in all cases where piles have been required with 2,000-lb. concrete they have been used also with the 3,000-lb. concrete, so that the difference in price is not greatly affected. In actual designing, cases may arise where the saving in dead weight will make it possible to use spread footings with 3,000-lb. concrete as against piles for 2,000-lb. concrete and so make large savings not shown in this comparison. With this information the proper unit cost for each item may be readily determined by an experienced estimator.

The unit costs which I have used in Table 144 and Fig. 18 are as follows:

Reinforcing bars in place $70.00 per ton.
Spiral, ties and stirrups in place $90.00 per ton.
Wood formwork, erected and removed, 25¢ per sq. ft.
Steel column forms, rental and labor, $15.00 per column per story.
Hand excavation, including backfill, $2.70 per cu. yd.
Concrete piles, 30 tons capacity, $42.00 each.

2,000-lb. concrete in place			$10.95 per cu. yd.
1.35 bbl. cement and sacks	at $2.75 =	$3.72	
0.59 cu. yd. fine aggregate	at 2.50 =	1.48	
0.70 cu. yd. coarse aggregate	at 2.50 =	1.75	
Water (7½ gal. per sack)			
Mixing and placing, including plant and other overhead charges and cost of protecting and curing	at 4.00		
3,000-lb. concrete in place			$11.66 per cu. yd.
1.65 bbl. cement and sacks	at $2.75 =	$4.54	
0.56 cu. yd. fine aggregate	at 2.50 =	1.40	
0.69 cu. yd. coarse aggregate	at 2.50 =	1.72	
Water (6 gal. per sack)			
Mixing and placing, including plant and other overhead charges and cost of protecting and curing	at 4.00		
5,000-lb. concrete in place			$13.47 per cu. yd.
2.4 bbl. cement and sacks	at $2.75 =	$6.60	
0.41 cu. yd. fine aggregate	at 2.50 =	1.02	
0.74 cu. yd. coarse aggregate	at 2.50 =	1.85	
Water (4½ gal. per sack)			
Mixing and placing, including plant and other overhead charges and cost of protecting and curing	at 4.00		

The quantities in the mixes given above are based on tests in which both gravel and crushed limestone were used for coarse aggregates and

in which the slumps varied from 5 in. to 7 in. The maximum size of the aggregate was ¾ in.

Advantages of Stronger Concrete.—Engineers are so accustomed to thinking of concrete for buildings in terms of 2,000-lb. strength at 28 days that it may be novel to consider using a 3,000-lb. concrete as the basic mix. For a great many years, however, it has been general practice to use 3,000-lb. concrete in the more heavily loaded columns of buildings. With the continually widening use of scientific concrete in recent years, many engineers and contractors have discovered that the full economy of scientific concreting can be secured in 2,000-lb. strength only at the sacrifice of workability. In order to keep a concrete of easy working quality on the job, contractors are generally—under usual weather conditions—providing a concrete of about 2,500-lb. strength in structures where only 2,000-lb. concrete is required. This experience has led many engineers to design structures in 2,500 or 3,000-lb. concrete in order to utilize the strength actually provided on the job. My own experience has led me to believe that a slight addition in cement (and a big reduction in water) as compared with the old-time practice, will produce a 3,000-lb. concrete which is more workable and less permeable at a very small increase in cost over the 2,000-lb. variety Even if such a concrete should cost as much in terms of the completed building as 2,000-lb. concrete, it would be preferable for several reasons. The increased workability due to added cement (as compared with 2,000-lb. concrete) will facilitate placing and surfacing and so overcome the objection of the field forces to stiff consistencies. The added cement and decreased water will greatly decrease the porosity and permeability and increase the weather resistance of the concrete. Such concrete will attain strength more rapidly and as a result will permit an earlier finishing of the work and require protection against low temperatures for a shorter period. It will also afford a much better bond with the reinforcing steel, and thus permit the use of larger bar sizes and shorter bars. It will decrease the thickness of members as controlled by compression and diagonal tension and decrease the weight of the supporting structure. The shrinkage, while greater than that for 2,000-lb. concrete, will be considerably less than that of the old-time wet 1 : 2 : 4 concrete. In the case of 5,000-lb. concrete as used in heavily loaded columns, the actual amount of cement and the shrinkage will be less than with the old-time wet 1 : 1 : 2 (3,000-lb.) concrete. This shrinkage consideration will limit the use of *exceedingly* rich mixes, especially in slabs and walls of great length, but it will not deter from the use of properly made 3,000-lb. concrete in floors and walls or of properly made 5,000-lb. concrete in columns, since it involves less shrinkage than existing buildings are carrying comfortably.

Flat Slab Cost Study.—A study of tables Nos. 145 and 146 or of 147 and 148 shows that there is but a small difference in cost as between a four-way flat slab floor (or roof) made of 2,000-lb. or 3,000-lb. concrete. While these quantities are based on actual computations by the four-way system, the provisions of the Code are such that no material differences would be expected from computations by the two-way system. At the unit prices given before the saving with stronger concrete is from 1.0 to 1.8 cents per sq. ft. of floor area. The real saving is secured in the columns and footings through the reduction in dead weight. The quantities in tables Nos. 153 and 154 are priced and plotted in Fig. 18. A study of the data indicates in a general way the following comparisons for complete buildings:

(1) A considerable saving in cost for all loads and spans may be effected by the use of 3,000-lb. concrete.

(2) This saving increases (but at a decreasing rate of increase) with the number of stories in the building.

(3) This saving increases as the soil pressure decreases.

(4) This saving increases slightly with larger panel sizes.
(5) This saving increases markedly with heavier live loads.

The least saving of the 3,000-lb. over the 2,000-lb. concrete (smaller panel, lighter live load, higher soil pressure) amounts to 1.8 cents per sq. ft. of total floor area for a six-story building and increases to 3.3 cents per sq. ft. for a twelve-story building. The greatest saving (larger panel,

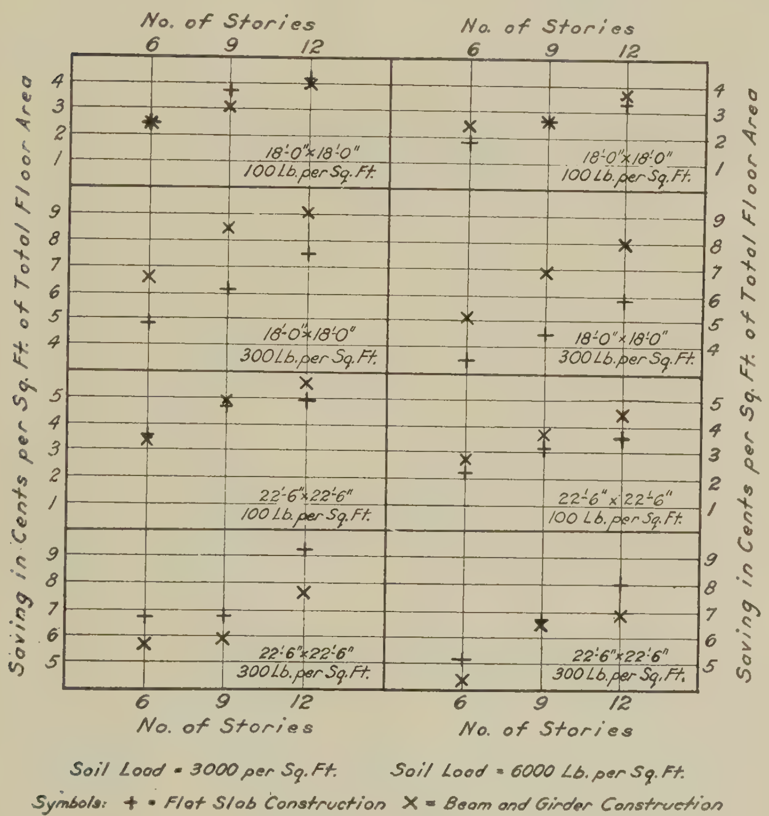


FIG. 18.—SAVING FROM USE OF 3,000-LB. CONCRETE.

heavier live load, lower soil pressure and higher building) amounts to 9.2 cents per sq. ft. for a twelve-story building. The maximum saving, expressed in percentage of the cost of the concrete frame is over 7 per cent. The least saving is over 2 per cent.

Beam-and-Girder Cost Study.—In this study of the "beam-and-girder" system the framing of the members was as follows: Girders from column to column in one direction, beams directly from column to column in the other direction, two beams (at the third points) between these marginal beams and one-way slabs spanning between all beams. The slabs have temperature and shrinkage reinforcement as required by the Joint Code. A

study of Tables 149 and 150, or of 151 and 152 shows that there is an appreciable saving in cost in the floor and roof construction itself by the use of the 3,000-lb. concrete. At the unit prices given before this saving amounts to from 1.6 to 3.5 cents per square foot. There is also a reduction in the dead weight of the construction, resulting in further savings in the cost of colums and foundations. The quantities of Tables 155 and 156 are priced and plotted in Fig. 18. A study of these data indicates in a general way for complete buildings:

(1) A considerable saving for all loads and spans may be effected by the use of 3,000-lb. concrete.

(2) This saving increases (but at a decreasing rate) with the number of stories in the building.

(3) This saving increases as the soil pressure decreases.

(4) This saving increases as the panel size increases with 100-lb. live-load, but decreases as the panel size increases with 300-lb. live-load.

(5) This saving increases markedly with heavier live loads.

The least saving (smaller panel, lighter live load, higher soil pressure) amounts to 2.5 cents per sq. ft. of total floor area for a six-story building and increases to 4.0 cents per sq. ft. for a twelve-story building. The greatest saving (smaller panel, heavier live load, lower soil pressure and higher building) amounts to 9.1 cents per sq. ft. for a twelve-story building. Expressed in percentage of cost of the concrete frame, the maximum saving is 7.2 per cent. The least saving is 3.1 per cent.

Comparison of Flat Slab and Beam-and-Girder Construction.—In general the savings effected by the use of 3,000-lb. concrete are greater with beam-and-girder construction than with flat slab. An exception occurs in the case of the larger panel with the heavier live load. The total cost of the beam-and-girder construction is always greater than that of the corresponding flat slab. The flat slab type, as here designed, is the standard form with enlarged column capitals and drop panels at the column head. Where the capital or the dropped panel, or both, must be omitted for architectural reasons the cost of the flat slab type would be greatly increased.

A study of the data of Table 144 leads to the following general conclusions as to the excess in cost of beam-and-girder over standard flat slab construction:

(1) It becomes greater as the load increases.
(2) It becomes greater as the soil pressure increases.
(3) It becomes less as the panel size increases.
(4) It is not greatly affected by the number of stories.

Expressed in figures the beam-and-girder type costs about 9.5 per cent more for the 18-ft. panel, 100-lb. *LL* and 3,000-lb. soil. This difference increases to 16 per cent for the 18-ft. panel, 3,000-lb. *LL* and 6,000-lb. soil. With 22-ft. 6-in. panel the first figure is reduced to about 3.5 per cent and the second to about 10 per cent.

Comparison With Chicago Code.—I have in my office complete quantities for designs in accordance with the present Chicago Code (now in process of revision) which afford comparisons between the cost of the joint requirements and those of a well-known and widely-used present standard. The great length of this paper forbids anything beyond a very brief statement of this comparison.

For the four-way flat slab type the cost is nearly identical on the basis of 2,000-lb. concrete. The Joint Code, however, will give a considerably better balanced design than the Chicago ordinance. The increased stresses in Joint Code are offset by added anchorage of reinforcement and other safeguards.

For the beam-and-girder type the cost in accordance with the Joint

Code is about 8 per cent less than the Chicago figure. This saving is for a typical interior panel and the saving for the entire building including exterior panels would be somewhat less. The Joint Code requires far better anchorage of reinforcement than the Chicago Code but the clear span moment calculations and the increased stresses more than offset this addition.

Most designers, I feel, will agree that the effect secured by the Joint Code in decreasing the difference in cost between the beam-and-girder and the flat-slab construction is desirable. The present advantage enjoyed by the flat-slab type will still remain but will be decreased in amount by the adoption of the Joint Code.

In the Chicago beam-and-girder designs no shrinkage and temperature reinforcement was included.

Savings from Scientific Concrete Proportioning.—All of the figures in this paper are based on the use of moderately stiff, non-segregating concrete. Such concrete may be manufactured and placed in the field to give any required strength at a considerably less cost than the wet consistencies often used in the past. This added saving acts to increase the final saving that may be realized by the use of 3,000-lb. scientific concrete to values greater than shown in this paper if comparison is made with the cost of old-time sloppy concrete of the same strength.

Use of Cost Data for Estimating.—The data of Tables 145 to 156 inclusive may be useful to engineers in estimating in advance the cost of proposed buildings to be designed under the 1928 Joint Code. In making use of this material for this purpose it is advisable to comply with the following necessary limitations:

(1) Determine proper cost units for the conditions of the contemplated work. The unit costs as given in this paper and on which Table 71 and Fig. 18 are based are not applicable, except by coincidence, to such use. They are too large for large jobs and too small for small jobs. The size of building, number of re-uses of forms, cost of materials and labor, weather conditions, and many local conditions will determine the proper values of cost units in any case.

(2) Proper allowance must be made for exterior and special panels, since these tables deal with typical interior panels exclusively.

(3) Only the pit excavation for footings is included under the item of excavation in the table. The general (steam shovel) excavation must be added, with allowance for overrun at sides of lot.

(4) A small allowance should be made for necessary wastage of material in the final design. In these tables fractional bars have been used occasionally in order to permit of accurate interpolation between the computed values. In a few cases the soil pressure must slightly exceed that given in the tables in order to keep the footings within the panel area.

(5) Column live loads on all floors have been reduced in accordance with Chicago practice—15 per cent for the top floor, 20 per cent for the next, increasing by 5 per cent to 50 per cent for the eighth floor from the top and for all lower floors. The Joint Code does not prescribe any load reductions, since this would be a general ordinance provision applying to concrete in common with all other types of building construction. Allowance must be made in case other load reduction provisions are in force.

(6) The size of columns will affect the quantities markedly. Allowance must be made if very small columns are required. In these designs the diameter or side of the top-story column has been made about one-twelfth of the panel dimension, and the percentage of column vertical reinforcement has been kept in the 2 to 3 per cent region generally. Stronger concrete has been used as the load increases, rather than very high steel percentages.

(7) If piles or caissons are required, the cost will be considerably

increased, except where piles are shown in the tables. Where piles are shown allowance must be made if the depth of pile cut-off requires additional excavation, unless gravity footings are possible and effect a balancing saving.

(8) In interpolating between the panel sizes and loadings covered by my computations the *trend* of the data as given in a diagram similar to Fig. 18 drawn with the proper unit prices for the work in hand, should be carefully studied.

(9) Joist and girder type, or beam and girder with other than third point beam spacing, will affect the quantities, and proper allowance should be made for the actual framing to be used.

(10) If the flat slab capital or depressed panel is to be different from the values given in Diagrams 77, 79, etc., in Part I of this paper, allowance must be made for the effect of such variation in increasing or decreasing quantities.

TABLE 144.—COST IN CENTS PER SQUARE FOOT OF TOTAL FLOOR AREA.

Side of Square Panel	Live Load, lb.	Soil Load, lb.	Concrete Strength, lb. per sq. in.	Flat Slab			Beam and Girder		
				6 Story	9 Story	12 Story	6 Story	9 Story	12 Story
18 ft. 0 in.	100	3000	2000	72.7	76.4	79.7	80.6	83.8	87.6
18 ft. 0 in.	100	3000	3000	70.2	72.7	75.5	78.1	80.7	83.6
			Saving	**2.5**	**3.7**	**4.2**	**2.5**	**3.1**	**4.0**
18 ft. 0 in.	100	6000	2000	69.4	71.9	74.0	77.8	80.5	83.3
18 ft. 0 in.	100	6000	3000	67.6	69.2	70.7	75.2	77.8	79.6
			Saving	**1.8**	**2.7**	**3.3**	**2.6**	**2.7**	**3.7**
18 ft. 0 in.	300	3000	2000	94.7	102.1	108.7	110.0	119.0	126.4
18 ft. 0 in.	300	3000	3000	89.9	96.0	101.2	103.4	110.5	117.3
			Saving	**4.8**	**6.1**	**7.5**	**6.6**	**8.5**	**9.1**
18 ft. 0 in.	300	6000	2000	84.9	90.7	95.6	101.1	108.5	114.6
18 ft. 0 in.	300	6000	3000	81.4	86.2	89.7	96.0	101.6	106.6
			Saving	**3.5**	**4.5**	**5.9**	**5.1**	**6.9**	**8.0**
22 ft. 6 in.	100	3000	2000	82.6	87.2	91.5	86.0	90.4	94.2
22 ft. 6 in.	100	3000	3000	79.0	82.5	86.6	82.7	85.5	88.6
			Saving	**3.6**	**4.7**	**4.9**	**3.3**	**4.9**	**5.6**
22 ft. 6 in.	100	6000	2000	76.7	80.2	83.4	82.2	85.3	88.5
22 ft. 6 in.	100	6000	3000	74.5	77.1	79.8	79.4	81.5	83.9
			Saving	**2.2**	**3.1**	**3.6**	**2.8**	**3.8**	**4.6**
22 ft. 6 in.	300	3000	2000	110.2	125.7	131.8	116.8	136.1	143.9
22 ft. 6 in.	300	3000	3000	103.4	118.9	122.6	111.1	130.2	136.2
			Saving	**6.8**	**6.8**	**9.2**	**5.7**	**5.9**	**7.7**
22 ft. 6 in.	300	6000	2000	96.5	103.2	109.3	105.6	113.2	120.0
22 ft. 6 in.	300	6000	3000	91.4	97.5	101.3	101.3	107.6	113.1
			Saving	**5.1**	**5.7**	**8.0**	**4.3**	**5.6**	**6.9**

TABLE 145.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
1	Roof	155	..	381	..	330	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
2	Floor	193	..	506	..	331	..	..
3	1–6 Story Columns	113	64	1266	439	..	6	..
4	3000-lb. Footing	161	7	591	..	44	..	329
5	6000-lb. Footing	82	3	350	..	28	..	144
6	1–9 Story Columns	113	180	2742	804	..	9	..
7	3000-lb. Footing	293	15	1156	..	71	..	643
8	6000-lb. Footing	141	4	595	..	43	..	250
9	1–12 Story Columns	113	314	4868	1288	..	12	..
10	3000-lb. Footing	440	27	1820	..	90	..	1078
11	6000-lb. Footing	205	4	946	8	48	..	360
	300-LB. LIVE LOAD ON ALL FLOORS.							
12	Floor	236	..	855	..	333	..	..
13	1–6 Story Columns	57	144	2458	641	..	6	..
14	3000-lb. Footing	401	24	1695	..	82	..	960
15	6000-lb. Footing	198	5	883	..	44	..	338
16	1–9 Story Columns	57	300	5538	1294	..	9	..
17	3000-lb. Footing	687	47	2970	..	122	..	1804
18	6000-lb. Footing	339	8	1593	15	62	..	616
19	1–12 Story Columns	57	491	9668	2156	..	12	..
20	3000-lb. Footing	1028	76	4472	..	157	..	2740
21	6000-lb. Footing	507	12	2260	20	81	..	941

TABLE 146.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
22	Roof...............	..	136	..	407	..	329	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
23	Floor...............	..	170	..	552	..	330	..	..
24	1-6 Story Columns...	85	..	92	870	361	..	6	..
25	3000-lb. Footing.....	..	133	..	692	..	38	..	261
26	6000-lb. Footing.....	..	70	..	451	..	25	..	117
27	1-9 Story Columns...	85	..	208	1733	604	..	9	..
28	3000-lb. Footing.....	..	224	..	1240	..	55	..	473
29	6000-lb. Footing.....	..	112	..	629	..	32	..	194
30	1-12 Story Columns..	85	..	352	2910	925	..	12	..
31	3000-lb. Footing.....	..	337	..	2063	..	77	..	816
32	6000-lb. Footing.....	..	189	..	933	10	40	..	280
	300-LB. LIVE LOAD ON ALL FLOORS								
33	Floor...............	..	204	..	992	..	332	..	..
34	1-6 Story Columns...	57	28	116	1752	512	..	6	..
35	3000-lb. Footing.....	..	372	..	1782	..	77	..	740
36	6000-lb. Footing.....	..	148	4	855	..	39	..	256
37	1-9 Story Columns...	57	28	272	3568	938	..	9	..
38	3000-lb. Footing.....	..	591	41	3316	..	97	..	1390
39	6000-lb. Footing.....	..	271	6	1569	12	53	..	475
40	1-12 Story Columns..	57	28	463	6041	1487	..	12	..
41	3000-lb Footing.....	..	787	67	5018	..	123	..	2166
42	6000-lb. Footing.....	..	391	8	2291	15	67	..	698

TABLE 147.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

USUAL DESIGN—2000 LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
43	Roof................	303	..	810	..	516	..	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
44	Floor...............	378	..	1028	..	517	..	..	..
45	1–6 Story Columns...	197	108	2282	600	..	6	..	..
46	3000-lb. Footing.....	365	20	1490	..	76	..	842	..
47	6000-lb. Footing.....	173	4	624	..	43	..	299	..
48	1–9 Story Columns...	197	295	5144	1208	..	9	..	..
49	3000-lb. Footing.....	658	40	2784	..	109	..	1612	..
50	6000-lb. Footing.....	313	5	1453	11	60	..	535	..
51	1–12 Story Columns..	197	503	9216	2046	..	12	..	..
52	3000-lb. Footing.....	987	71	4250	..	153	..	2620	..
53	6000-lb. Footing.....	474	16	2216	25	83	..	925	..
	300-LB. LIVE LOAD ON ALL FLOORS.								
54	Floor...............	464	..	1701	..	520	..	..	..
55	1–6 Story Columns...	98	236	4279	954	..	6	..	..
56	3000-lb. Footing.....	852	58	3634	..	134	..	2210	..
57	6000-lb. Footing.....	421	9	1894	15	70	..	780	..
58	1–9 Story Columns...	98	470	9831	2054	..	9	..	..
59	3000-lb. Footing.....	500	25	2540	50	115	..	1005	28
60	6000-lb. Footing.....	716	20	3300	65	104	..	1452	..
61	1–12 Story Columns..	98	760	17291	3509	..	12	..	..
62	3000-lb. Footing.....	825	29	3493	110	147	..	1560	38
63	6000-lb. Footing.....	1086	35	4940	130	141	..	2252	..

TABLE 148.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
64	Roof.............	..	264	..	882	..	514	..	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.									
65	Floor	..	330	..	1135	..	516	..	..	..
66	1–6 Story Columns.	197	..	108	1826	536	..	6	..	..
67	3000-lb. Footing...	..	287	..	1640	..	69	..	601	..
68	6000-lb. Footing...	..	140	..	798	..	39	..	236	..
69	1–9 Story Columns.	197	..	295	3721	977	..	9	..	..
70	3000-lb. Footing...	..	499	..	3026	..	87	..	1153	..
71	6000-lb. Footing...	..	246	5	1480	..	50	..	420	..
72	1–12 Story Columns	197	..	503	6107	1495	..	12	..	..
73	3000-lb. Footing...	..	749	72	4957	..	129	..	2160	..
74	6000-lb. Footing...	..	344	9	2045	..	65	..	621	..
	300-LB. LIVE LOAD ON ALL FLOORS.									
75	Floor.............	..	396	..	1942	..	518	..	..	..
76	1–6 Story Columns.	98	49	187	3008	735	..	6	..	..
77	3000-lb. Footing...	..	694	..	3979	..	114	..	1725	..
78	6000-lb. Footing...	..	312	8	1866	15	62	..	574	..
79	1–9 Story Columns.	98	49	421	6458	1445	..	9	..	..
80	3000-lb. Footing...	..	450	22	2400	35	106	..	881	26
81	6000-lb. Footing...	..	545	18	3303	46	89	..	1122	..
82	1–12 Story Columns	98	49	710	11051	2376	..	12	..	..
83	3000-lb. Footing...	..	660	11	2328	32	119	..	1144	36
84	6000-lb. Footing...	..	825	30	4909	78	119	..	1679	..

TABLE 149.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
1	Roof	140	..	563	53	471	..
	100-LB. LIVE LOAD ON ALL FLOORS.						
2	Floor	147	..	738	79	471	..
3	1–6 Story Columns	75	87	1031	408	411	..
4	3000-lb. Footing	144	..	489	..	39	255
5	6000-lb. Footing	70	..	275	..	26	115
6	1–9 Story Columns	75	212	2268	738	669	..
7	3000-lb. Footing	244	..	870	..	55	484
8	6000-lb. Footing	117	4	465	..	35	206
9	1–12 Story Columns	75	368	4112	1190	957	..
10	3000-lb. Footing	362	20	1421	..	76	844
11	6000-lb. Footing	173	4	798	..	43	304
	300-LB. LIVE LOAD ON ALL FLOORS.						
12	Floor	190	..	1136	190	541	..
13	1–6 Story Columns	50	159	2500	685	480	..
14	3000-lb. Footing	362	20	1488	..	76	845
15	6000-lb. Footing	168	4	792	..	43	294
16	1–9 Story Columns	50	340	5680	1404	798	..
17	3000-lb. Footing	643	41	2699	..	108	1594
18	6000-lb. Footing	298	13	1429	..	65	613
19	1–12 Story Columns	50	568	10020	2356	1155	..
20	3000-lb. Footing	948	67	4078	..	142	2500
21	6000-lb. Footing	448	34	2125	..	95	1045

TABLE 150.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
22	Roof	..	127	..	466	26	476	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
23	Floor	..	135	..	635	51	475	..
24	1–6 Story Columns	75	25	62	872	379	411	..
25	3000-lb. Footing	..	114	..	556	..	34	202
26	6000-lb. Footing	..	59	..	298	..	24	98
27	1–9 Story Columns	75	25	188	1474	585	699	..
28	3000-lb. Footing	..	197	..	1008	..	47	356
29	6000-lb. Footing	..	102	..	483	..	32	171
30	1–12 Story Columns	75	25	343	2426	852	957	..
31	3000-lb. Footing	..	296	..	1631	..	64	664
32	6000-lb. Footing	..	126	..	762	..	37	236
	300-LB. LIVE LOAD ON ALL FLOORS							
33	Floor	..	169	..	925	152	548	..
34	1–6 Story Columns	50	..	159	1727	540	480	..
35	3000-lb. Footing	..	296	..	1699	..	64	660
36	6000-lb. Footing	..	145	..	786	..	39	242
37	1–9 Story Columns	50	..	340	3720	1026	798	..
38	3000-lb. Footing	..	512	..	2984	..	93	1250
39	6000-lb. Footing	..	246	5	1405	..	51	421
40	1–12 Story Columns	50	..	568	6590	1667	1155	..
41	3000-lb. Footing	..	775	..	4802	..	124	2000
42	6000-lb. Footing	..	352	9	2285	..	65	651

TABLE 151.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

USUAL DESIGN—2000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete		Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
43	Roof	230	..	1043	81	739	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.							
44	Floor	250	..	1219	143	796	..	..
45	1–6 Story Columns	114	131	1869	573	524	..	..
46	3000-lb. Footing	266	13	1017	..	60	574	..
47	6000-lb. Footing	131	4	588	..	36	230	..
48	1–9 Story Columns	114	313	4241	1138	843	..	..
49	3000-lb. Footing	485	26	1976	..	88	1120	..
50	6000-lb. Footing	222	5	1037	..	50	388	..
51	1–12 Story Columns	114	543	7630	1893	1201	..	..
52	3000-lb. Footing	722	47	2986	..	119	1810	..
53	6000-lb. Footing	348	7	1603	..	68	614	..
	300-LB. LIVE LOAD ON ALL FLOORS.							
54	Floor	311	..	2208	340	846	..	..
55	1–6 Story Columns	78	228	4144	1015	587	..	..
56	3000-lb. Footing	685	48	2953	..	116	1793	..
57	6000-lb. Footing	330	10	1585	..	64	600	..
58	1–9 Story Columns	78	496	9425	2153	979	..	..
59	3000-lb. Footing	547	62	2135	..	225	953	25
60	6000-lb. Footing	590	20	2710	..	90	1166	..
61	1–12 Story Columns	78	857	16438	3654	1435	..	..
62	3000-lb. Footing	787	243	3365	..	340	1798	33
63	6000-lb. Footing	903	35	4096	..	120	1830	..

TABLE 152.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

DESIGN USING 3000-LB. CONCRETE, EXCEPT IN COLUMNS.

Item No.	Item	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
		2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
64	Roof	..	207	..	846	59	747	..	..
	100-LB. LIVE LOAD ON ALL FLOORS.								
65	Floor	..	220	..	1092	96	804	..	..
66	1-6 Story Columns	114	38	93	1564	508	524	..	..
67	3000-lb. Footing	..	219	..	1206	..	52	435	..
68	6000-lb. Footing	..	110	..	597	..	33	184	..
69	1-9 Story Columns	114	38	275	2755	840	843	..	..
70	3000-lb. Footing	..	378	..	2042	..	75	832	..
71	6000-lb. Footing	..	182	..	966	..	42	305	..
72	1-12 Story Columns	114	38	505	4574	1291	1201	..	..
73	3000-lb. Footing	..	569	..	3183	..	102	1372	..
74	6000-lb. Footing	..	269	..	1547	..	53	452	..
	300-LB. LIVE LOAD ON ALL FLOORS.								
75	Floor	..	289	..	1931	283	855	..	..
76	1-6 Story Columns	78	41	188	3216	804	587	..	..
77	3000-lb. Footing	..	572	..	3396	..	99	1431	..
78	6000-lb. Footing	..	268	6	1596	..	54	470	..
79	1-9 Story Columns	78	41	455	6688	1584	979	..	..
80	3000-lb. Footing	..	532	..	2048	..	200	866	24
81	6000-lb. Footing	..	464	13	2830	..	80	888	..
82	1-12 Story Columns	78	41	816	11261	2504	1435	..	..
83	3000-lb. Footing	..	821	..	3135	..	295	1535	32
84	6000-lb. Footing	..	706	22	4252	..	106	1416	..

TABLE 153.—FLAT SLAB QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL. TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units	
101	6	100	3000	2000	1394	71	..	4768	439	2029	6	329
102	6	100	3000	3000	85	1119	92	4729	361	2017	6	261
103	9	100	3000	2000	2105	195	..	8327	804	3049	9	643
104	9	100	3000	3000	85	1720	208	7796	604	3024	9	473
105	12	100	3000	2000	2831	341	..	12635	1288	4061	12	1078
106	12	100	3000	3000	85	2343	352	11452	925	4036	12	816
107	6	100	6000	2000	1315	67	..	4537	439	2013	6	144
108	6	100	6000	3000	85	1056	92	4488	361	2004	6	117
109	9	100	6000	2000	1953	184	..	7781	804	3021	9	250
110	9	100	6000	3000	85	1608	208	7185	604	3001	9	194
111	12	100	6000	2000	2596	318	..	11761	1296	4019	12	360
112	12	100	6000	3000	85	2173	352	10260	911	3999	12	280
113	6	300	3000	2000	1793	168	..	8809	641	2077	6	960
114	6	300	3000	3000	57	1506	116	8937	512	2066	6	740
115	9	300	3000	2000	2787	347	..	15729	1294	3116	9	1760
116	9	300	3000	3000	57	2321	313	15227	938	3082	9	1390
117	12	300	3000	2000	3836	567	..	23899	2156	4150	12	2740
118	12	300	3000	3000	57	3195	530	22378	1487	4104	12	2166
119	6	300	6000	2000	1590	149	..	8004	641	2039	6	338
120	6	300	6000	3000	57	1332	120	8010	512	2028	6	256
121	9	300	6000	2000	2439	308	..	14352	1309	3056	9	616
122	9	300	6000	3000	57	2067	278	13408	950	3038	9	475
123	12	300	6000	2000	3315	503	..	21741	2176	4074	12	941
124	12	300	6000	3000	57	2799	471	19651	1502	4048	12	698

TABLE 154.—FLAT SLAB QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Formwork		Hand Excavation, cu. ft.	Number of Concrete Piles
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.	Wood, sq. ft.	Steel, units		
125	6	100	3000	2000	2755	128	..	9722	600	3177	6	842	..
126	6	100	3000	3000	197	2201	108	10017	536	3163	6	601	..
127	9	100	3000	2000	4182	335	..	16862	1208	4761	9	1612	..
128	9	100	3000	3000	197	3403	295	16709	977	4729	9	1153	..
129	12	100	3000	2000	5646	574	..	25584	2154	6356	12	2620	..
130	12	100	3000	3000	197	4643	575	24431	1495	6319	12	2160	..
131	6	100	6000	2000	2563	112	..	8856	600	3144	6	299	..
132	6	100	6000	3000	197	2054	108	9175	536	3133	6	236	..
133	9	100	6000	2000	3837	300	..	15631	1219	4712	9	535	..
134	9	100	6000	3000	197	3150	300	15163	977	4692	9	420	..
135	12	100	6000	2000	5132	519	..	23550	2071	6286	12	925	..
136	12	100	6000	3000	197	4238	512	21519	1495	6255	12	621	..
137	6	300	3000	2000	3573	294	..	17228	954	3250	6	2210	..
138	6	300	3000	3000	98	2987	187	17579	735	3218	6	1725	..
139	9	300	3000	2000	4613	495	..	26789	2104	4791	9	1005	28
140	9	300	3000	3000	98	3931	443	25276	1490	4764	9	881	26
141	12	300	3000	2000	6330	789	..	40305	3619	6383	12	1560	38
142	12	300	3000	3000	98	5329	721	35623	2408	6331	12	1144	36
143	6	300	6000	2000	3142	245	..	15488	969	3186	6	780	..
144	6	300	6000	3000	98	2605	195	15460	750	3166	6	574	..
145	9	300	6000	2000	4829	490	..	27509	2119	4780	9	1452	..
146	9	300	6000	3000	98	4026	439	26151	1498	4747	9	1122	..
147	12	300	6000	2000	6581	795	..	41752	3639	6377	12	2252	..
148	12	300	6000	3000	98	5486	740	38204	2454	6326	12	1679	..

TABLE 155.—BEAM AND GIRDER QUANTITIES—18 FT. BY 18 FT. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.		
101	6	100	3000	2000	1096	87	..	5773	856	3276	255
102	6	100	3000	3000	75	939	62	5069	660	3296	202
103	9	100	3000	2000	1637	212	..	9605	1423	4963	484
104	9	100	3000	3000	75	1426	188	8128	1019	4992	356
105	12	100	3000	2000	2197	388	..	14214	2112	6685	844
106	12	100	3000	3000	75	1929	343	11508	1439	6722	664
107	6	100	6000	2000	1022	87	..	5559	856	3263	115
108	6	100	6000	3000	75	884	62	4811	660	3286	98
109	9	100	6000	2000	1510	216	..	9200	1423	4943	206
110	9	100	6000	3000	75	1331	188	7505	1019	4977	171
111	12	100	6000	2000	2008	372	..	13591	2112	6652	304
112	12	100	6000	3000	75	1759	343	10639	1473	6695	236
113	6	300	3000	2000	1502	179	..	10231	1688	3732	845
114	6	300	3000	3000	50	1268	159	8517	1326	3760	660
115	9	300	3000	2000	2353	381	..	18030	2977	5705	1594
116	9	300	3000	3000	50	1991	340	14570	2242	5751	1250
117	12	300	3000	2000	3228	635	..	27157	4499	7719	2500
118	12	300	3000	3000	50	2761	568	22033	3365	7763	2000
119	6	300	6000	2000	1308	163	..	9535	1688	3699	294
120	6	300	6000	3000	50	1117	159	7604	1326	3735	242
121	9	300	6000	2000	2008	353	..	16760	2977	5662	613
122	9	300	6000	3000	50	1725	345	12991	2242	5709	421
123	12	300	6000	2000	2728	602	..	25204	4499	7672	1045
124	12	300	6000	3000	50	2338	577	19516	3365	7704	651

TABLE 156.—BEAM AND GIRDER QUANTITIES—22 FT. 6 IN. BY 22 FT. 6 IN. INTERIOR PANEL.

TOTAL QUANTITIES FOR ONE PANEL FROM FOOTING TO ROOF.

Item No.	Number of Stories	Live Load, lb. per sq. ft.	Soil Load, lb. per sq. ft.	Concrete, lb. per sq. in.	Concrete			Reinforcing Steel		Wood Forms, sq. ft.	Hand Excavation, cu. ft.	Number of Concrete Piles
					2000-lb., cu. ft.	3000-lb., cu. ft.	5000-lb., cu. ft.	Bars, lb.	Spiral, lb.			
125	6	100	3000	2000	1860	144	..	10024	1369	5303	574	..
126	6	100	3000	3000	114	1564	93	9070	1037	5343	435	..
127	9	100	3000	2000	2829	339	..	17012	2363	8038	1120	..
128	9	100	3000	3000	114	2383	275	14379	1667	8081	832	..
129	12	100	3000	2000	3816	590	..	25068	3547	10815	1810	..
130	12	100	3000	3000	114	3234	505	20615	2406	10872	1372	..
131	6	100	6000	2000	1725	135	..	9595	1369	5279	230	..
132	6	100	6000	3000	114	1455	93	8467	1037	5314	184	..
133	9	100	6000	2000	2566	318	..	16073	2363	8000	388	..
134	9	100	6000	3000	114	2187	275	13303	1667	8048	305	..
135	12	100	6000	2000	3442	550	..	23685	3547	10764	614	..
136	12	100	6000	3000	114	2934	505	18979	2406	10845	452	..
137	6	300	3000	2000	2548	276	..	19180	2796	5672	1793	..
138	6	300	3000	3000	78	2265	198	17113	2278	5708	1431	..
139	9	300	3000	2000	3343	558	..	30267	4954	8711	953	25
140	9	300	3000	3000	78	3092	455	25030	3907	8766	866	24
141	12	300	3000	2000	4516	1100	..	45134	7475	11820	1798	33
142	12	300	3000	3000	78	4248	816	36483	5676	11882	1535	32
143	6	300	6000	2000	2193	238	..	17822	2796	5620	676	..
144	6	300	6000	3000	78	1961	194	15313	2278	5663	470	..
145	9	300	6000	2000	3386	516	..	30842	4954	8576	1166	..
146	9	300	6000	3000	78	3024	468	25812	3907	8648	888	..
147	12	300	6000	2000	4632	892	..	45865	7475	11600	1830	..
148	12	300	6000	3000	78	4133	838	37600	5676	11693	1416	..

Discussion, Design and Cost Data, 1928 Building Code.

Herbert J. Gilkey* and William H. Thoman.†

During the past two years the writers have been devoting some attention to the subject of simplified design aids in the field of reinforced concrete. It is natural, therefore, that material of the sort presented by Mr. Lord should be of great interest to them. There is also the possibility that their own efforts might add something of value to this splendid project. Several phases of the subject have received attention but the present discussion will be restricted to columns. **Mr. Gilkey and Mr. Thoman**

Two charts are presented herewith, Fig. 1 and Fig. 2. They cover the usual column range, both spiralled and tied and all current steel and concrete stresses. They are applicable to all recognized tied column specifications and to those spiralled column formulas that allow for the effect of spiral by permitting a higher f_c value. These include the 1928 A. C. I., 1916 and 1924 Joint Committee and all similar specifications. With slight improvising the column chart may be adapted to such as the New York and Chicago codes as regards spiralled column design.

These charts are not offered with any claim of superiority but rather to present the matter in a slightly different form that may be preferred by some and rejected by others. The discussion will be restricted to an attempt to point out the main differences without bias.

Since, under a single specification such as the A. C. I. code, the concrete working stresses and strength of the concrete bear a fixed relationship to one another, it is feasible for Mr. Lord to base his charts upon representative concretes as indicated by the ultimate strengths. Moreover, it is possible for him to include the spiral and longitudinal steel both on the same chart.

Since under different codes the same concrete will be used with different factors of safety (i. e. have varied working stresses) in a general chart account must be taken of the actual working stress with its corresponding values of n and p. The f_c and n_p axes accomplish this for all concretes and steel percentages. The table on the chart, Fig. 1, gives value of f_c and n corresponding to different steel ratios and ultimate concrete strengths for the 1928 A. C. I. and 1924 Joint Committee Codes for both tied and spiralled columns. The working stresses corresponding to given grades of concrete by other codes must be obtained from the provisions of the code in question.

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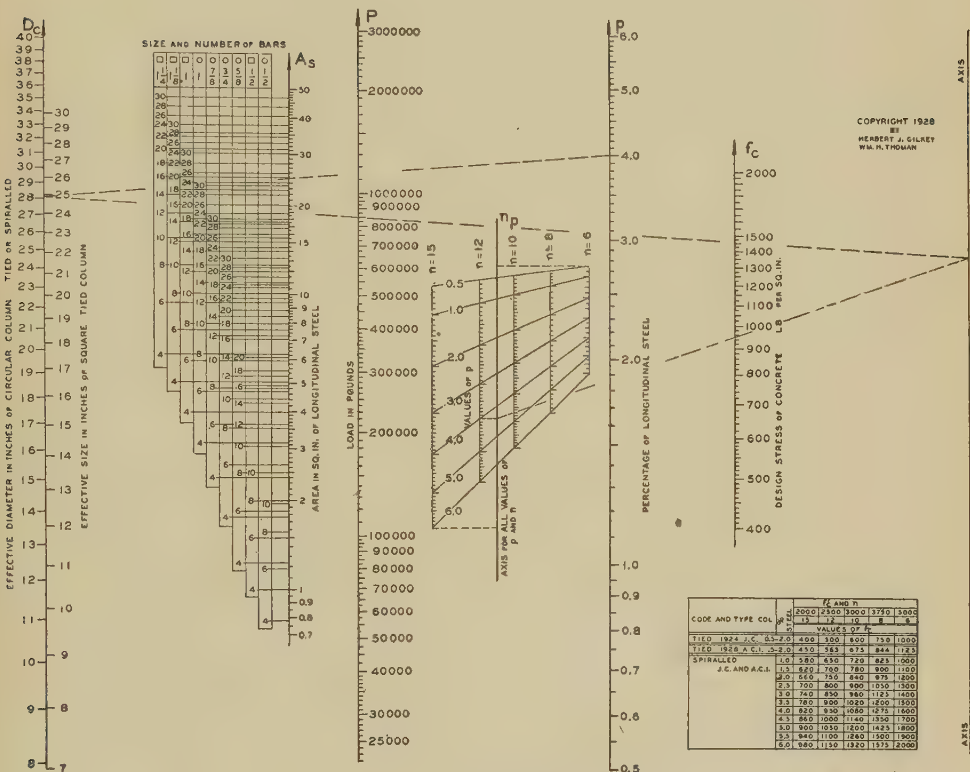
Under different codes there are many possible combinations of spiral and vertical steel and in the general case it therefore seems advisable to have a separate chart for spiral design, Fig. 2, which will fit any specification regardless of the vertical steel. In a one-specification chart the separation is not desirable and Mr. Lord's method is preferable in his case since the A. C. I. code is the only one considered by him.

Mr. Lord has kept in view the fact that any diagram or chart, to be generally used, must appear to be very simple. In doing this he has omitted one or two features that would add to their useability without much added congestion. To begin the design one assumes the size of column and divides this into the load for the value of P/A. After assuming D, A must be computed or else obtained from some table such as Table 105. If areas were added at the left of the right-hand line (diagrams 94-99) it would not be necessary to leave the chart to find A. By the addition of another axis, P/A could be found directly on the alignment chart. This latter would detract from the simplicity and was probably omitted by intent on that account. On the writers' charts A is not shown but it is not needed as such.

The steel selection offered by Mr. Lord is somewhat less both as to longitudinal and spiral steel but in most cases this would not be deemed objectionable. There is some overlapping of spiral sizes in the vicinity of maximum spacing that does not show, one spiral size ending where the next begins. This again seems fully justified in the interests of simplicity. A_s and percentage scales beside the two middle axes would often be useful but would again add to the congestion. The complete design is obtained without the designer knowing what percentage of steel he has used unless he turns to diagram 99. In most instances it is not vital that he know the percentages since any design obtained from the diagrams will fall within the limits of the A. C. I. Code.

There is one fundamental difference between the two types of charts. Mr. Lord assumes a column diameter and obtains P/A. If the P/A value is not feasible on the basis of the assumed diameter of column, he makes another assumption. For large buildings in which many columns of the same size are to be used, it is logical to let the percentage of reinforcement be the variable.

The writers' chart starts the design with P, p, n and f_c and solves directly for the diameter of column (or side of square tied column). This is a very convenient procedure for the less frequent designer and for different designs. If it be desired to hold a constant size of column but vary the reinforcement to meet the variation in column loads, the writers' type then becomes indirect. The procedure in such a case is to assume a new value for p. Enter the n_p line, and span past proper value for f_c (from table on chart or from code being followed) to axis. Span from axis to the D_c value that has been stipulated or assumed. Read load. If too high or too low as compared with actual, make a new assumption of p and repeat. Thus both kinds of charts have their cut-and-try element.



CODE AND TYPE COL	% STEEL	f'_c AND n: 2000 / 15	2500 / 12	3000 / 10	3750 / 8	5000 / 6
		VALUES OF f_c				
TIED 1924 J.C.	0.5–2.0	400	500	600	750	1000
TIED 1928 A.C.I.	.5–2.0	450	563	675	844	1125
SPIRALLED J.C. AND A.C.I.	1.0	580	650	720	825	1000
	1.5	620	700	780	900	1100
	2.0	660	750	840	975	1200
	2.5	700	800	900	1050	1300
	3.0	740	850	960	1125	1400
	3.5	780	900	1020	1200	1500
	4.0	820	950	1080	1275	1600
	4.5	860	1000	1140	1350	1700
	5.0	900	1050	1200	1425	1800
	5.5	940	1100	1260	1500	1900
	6.0	980	1150	1320	1575	2000

FIG. 1.—DESIGN CHART FOR REINFORCED-CONCRETE COLUMN.

Illustrative Problems:

Design spiralled column to support 850,000 lb. according to 1928 A. C. I. Code. Assume $f'_c = 2500$; $p = 0.04$; f_c and n (from table) = 950 and 12. Locate point on np axis horizontally opposite $p = 4$ and $n = 12$. Span from np to 950 on f_c. Rotate on axis and span to $P = 850{,}000$. On D_c read 28 in. core diameter. Span from D_c to $p = 4$ and read on A_s 16 1¼ in. square or 20 1⅛ in. square vertical bars required. For spiral use ½ in. dia. at 2¾ in. spacing. See illustrative problem on Fig. 2 for details of spiral selection.

Design tied column to support 300,000 lb. Assume $f'_c = 2500$; $P = 2$ per cent; f_c and n (from table) = 563 and 12. By above procedure: D_c = 21 in. square column or 23.5 in. round. Use 6 1¼ in. square or 8 1⅛ in. square bars.

NOTE.—Consult code used for such items as: f_c, limiting slenderness ratio, steel percentages, number and size of bars, fire proofing, size and spacing of ties, spirals, etc. In case non-standard bar sizes are used, select steel from scale on right of A_s. This chart is in no way limited to any particular specification or code.

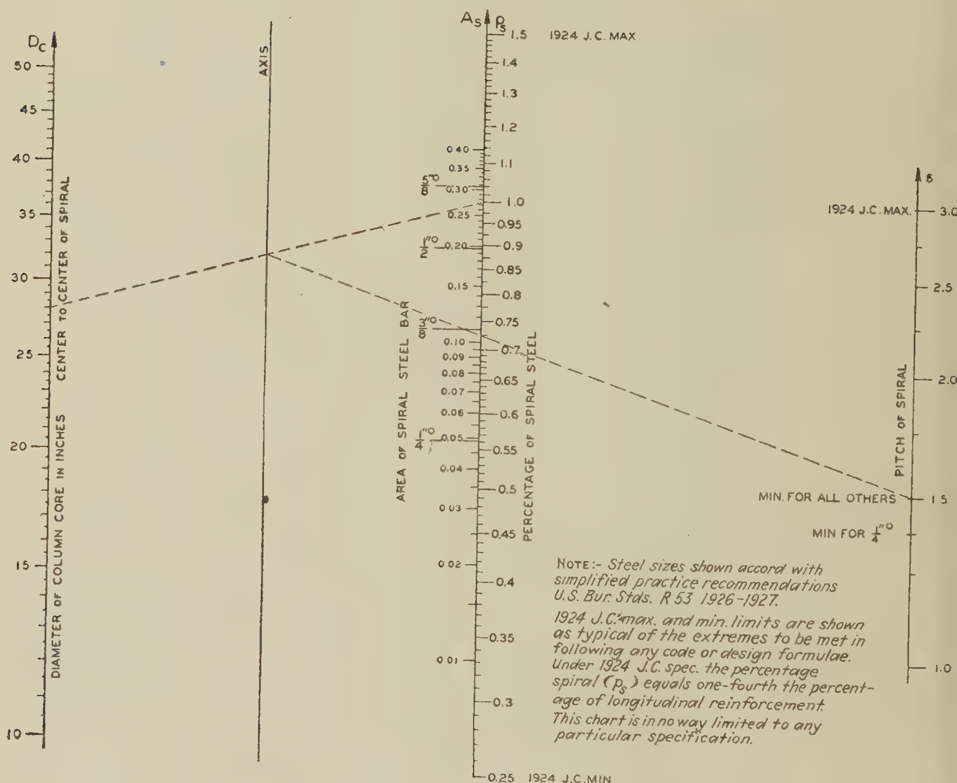


FIG. 2.—DESIGN CHART FOR SPIRAL REINFORCEMENT.

Illustrative Problem:

Known: Diameter of column core (D_C) = 28 in.
Percentage of spiral steel (p_S) = 1.0

Span from 28 on D_C to 1.0 on p_S. Pivot from axis to either spacing or size of spiral steel. (S or A_S) and select appropriate values. In this case at min. spacing of 1.5 in. a ⅜ in. dia. spiral would be used. In similar manner a ½ in. dia spiral at 2¾ in. spacing would be satisfactory. With very slight shortage of spiral area a ½ in. dia. at max. spacing of 3 in. would suffice and be selected by many designers.

For neither one is this a serious handicap. Either one may meet a certain set of design requirements or procedure better than the other.

The fact that the writers' charts are practically universal in their field of application is at present of some importance since general code agreement has not yet been attained.

To summarize Mr. Lord's treatment the following statements are applicable:

(a) It is designed for the 1928 A. C. I. code and that only (obviously and properly so).

(b) It includes seven separate charts (94-100 incl.) and his table 105. In any one design only one of the charts 94-98 inc. need be used in conjunction with his Diagram 99 and Table 105. For tied columns only diagram 100 need be consulted but only part of the design can be obtained from it.

(c) To start a design P is known and the column diameter must be assumed. Any value within the chart limits will conform to A. C. I. code requirements and the procedure is therefore very direct and simple in this respect.

(d) If column diameter be assumed (as recommended), P/A must be computed. For A, reference to a separate diagram or table such as Table 105 is necessary. It would seem worth while to add values for A opposite the diameters or better yet to add a line to the chart and get P/A directly without the necessity of computation or conscious evalution of A.

(e) What the percentage of steel (either spiral or longitudinal) may be is not apparent without consulting Diagram 99.

(f) In the steel as designated, the full possibilities for different sizes of spiral and longitudinal bars are not shown. This is not objectionable in most cases.

(g) Values of steel area appear only in terms of bars. Sometimes it is desirable to know A_s as such. It can, of course, be easily computed. An A_s scale could be added for longitudinal steel or both, with a slightly added congestion that is probably not warranted.

To summarize the writers' treatment, the following statements are applicable:

(a) It is directly applicable to several leading codes and with the exercise of slight ingenuity to most of the other well-known column formulas. The spiral chart is perfectly general and is independent of the longitudinal design.

(b) One chart for tied columns and this, with one more for spiralled columns furnishes the entire solution without recourse to other computations, tables or diagrams.

(c) To start a design, P, p, f_c and the assumed properties of concrete are known. Effective diameter of tied or spiral, or side of square tied column, is sought.

(d) If it be desired to hold the diameter constant and vary steel percentage to carry different loads, assume a value for p, span across f_c to

axis. Span from axis to stipulated D_c. If P intersection fails to agree closely with load to be carried, assume another value for p and repeat. In practice this can be done very rapidly. In designing for the diameter to carry a given load with a given steel ratio, the design is direct and involves no element of trial.

(e) Full possibilities of longitudinal and spiral steel sizes are shown, it is apparent what percentages are being used and an A_s scale appears in addition to bar listings.

(f) The fact that all grades of concrete (as indicated by f_c and n on chart proper and by f'_c as well for A. C. I. 1928 and 1924 Joint Committee in the auxiliary table on the chart) appear on the one chart facilitates ready comparison of different designs based upon different strengths of concrete as well as varied steel ratios. This is of value in comparative cost reconnaisance.

(g) The spiral chart, Fig. 2, is very simple in both appearance and use. The column chart, Fig. 1, is much less so in appearance, especially as compared with those of Mr. Lord. Actually its use is very direct and easy. Designing columns by either type of chart will effect a considerable economy of time and effort for many. Each set has its strong points and weaknesses but both represent an advance over many of the current design aids. Both Mr. Lord's work and the charts herewith submitted contain certain original features that are not necessarily limited in their application to this subject or to this phase of the subject. The purpose of the present discussion has not been primarily an effort to offer something better (although for certain applications some may consider these to be an improvement and others, probably the majority, will not). The intent has been rather to place before those interested auxiliary devices and ideas that will possibly widen the field of selection. The detailed comparisons may give also a better perspective in reaching decisions as to features to be adopted or rejected in the preparation of design aids for specific purposes.

Mr. Hatt. W. K. HATT.—I would like to know what the relation is between the working stresses and formulas as used by Mr. Lord, and the so-called joint committee report of the several engineering societies? Is there necessity for changing the joint committee report to make this document useful? I speak of this because, in the proposed building code of the Department of Commerce, it is stated that structures should be designed in accordance with the principles of the joint committee report.

Mr. Lord. A. R. LORD.—Answering Dr. Hatt, it would be necessary to change the joint committee report, especially in the matter of the steel stress, which is 20,000 lb. throughout these diagrams. Of course the steel stress does not enter into the stirrup spacing diagrams. But on the footings and columns, slabs and beams, you would have to use a 20,000-lb. steel stress instead of 18,000 lb. in order to use these diagrams. That is a change that came about since the joint committee report in 1924. It is in line with the general movement to reduce construction costs somewhat in all types—steel, wood and concrete.

Researches on Concrete Materials and on Plain and Reinforced Concrete.

Submitted by Committee E-3 on Research.

Introduction.—The activities of this committee during the past year have been confined chiefly to the assembling of information on investigations under way and the compilation of references to reports pertaining to concrete published during the year. This report follows the same general form as previous reports of the committee published in the 1926 and 1927 *Proceedings* of the Institute. The subject-matter of the report consists largely of brief summaries of the replies to questionnaires submitted to 215 testing laboratories in the United States and Canada and is classified as follows:

I. Researches on Cement,
II. " " Plain Concrete,
III. " " Plain Aggregate,
IV. " " Reinforced Concrete,
V. Suggested Researches on Concrete and Related Subjects,
VI. References to Papers and Reports Published during 1927.

The majority of investigations under way are concerned with plain concrete as was the case in previous years. Of the investigations on plain concrete, such subjects as methods of curing, methods of testing, and determination of the physical properties of concrete appear to be receiving the most attention, although investigations of the properties of cement and aggregates and of methods of testing these constituents of concrete are increasing in number. Investigations of reinforced concrete are directed toward the solution of problems in design of dams and arches, beams, and slabs. An important part of this year's report is the list of papers on concrete and related subjects which was compiled from engineering journals published in the United States and in foreign countries.

The committee expresses its appreciation to the many organizations and individuals who have co-operated by furnishing information for this report.

This report has been submitted to letter ballot of the committee, which consists of 12 members, of whom 10 have voted affirmatively, none negatively, and 2 have refrained from voting.

H. F. Gonnerman, *Chairman.*
F. E. Richart, *Secretary.*

Researches on Concrete Materials and on Plain and Reinforced Concrete.

Many of the researches listed below have not been completed and, in general, reports giving data of the tests are not available. In a few instances reference is made to the publication of reports. The conclusions given for certain of the investigations are those of the investigator and do not necessarily represent the views of the committee.

I. CEMENT.

Constitution of Portland Cement.—(Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C.). This is a continuation of an exhaustive chemical and physico-chemical study of portland cement and the pure compounds of which it is formed. Notable progress has made in phase equilibria studies. Other important phases of this investigation which have been finished or have approached completion are concerned with the complete hydrolysis of compounds occurring in portland cement, with the reaction of setting and hardening in these compounds, and with the action of various salt solutions on the reaction products.

Two-Inch Cubes as Test Specimens for Portland Cement.—(University of Maine, Orono). This investigation included tension and compression tests on 102 samples of portland cement of many different brands. Results described in Bulletin 19 of the Maine Technology Experiment Station.

Constitution and Properties of Portland Cement.—(Board of Water Supply, City of New York). A continuation of physical and chemical investigations of cement with a view to determining those characteristics which govern permanency of concrete, and its durability in service.

Tests of Portland Cements.—(Lehigh Portland Cement Co., Allentown, Pa., in co-operation with sub-committee VII of A. S. T. M. Committee C-1 on Cement). These tests were carried beyond those outlined by the subcommittee, and include a year's experimentation with fluid neat cement mixtures, and other types of test.

Development of Damp Closet for Cement Testing.—(Lehigh Portland Cement Co., Allentown, Pa.). The simplest solution seems to be a type of closet that provides for both moist air and flowing water storage, the latter eliminating need for special heating or refrigeration when the temperature of the entering water can be maintained at or near 70 deg. F. by an automatic valve or other temperature controlling device. This scheme has been found adequate to maintain the temperature of the closet well

within the specification limits through a round of seasons, and without insulation of the closet. Practically 100 per cent relative humidity can be maintained with an atomizer of the type supplied commercially to the textile industry.

Volume Changes in Neat Cement Pastes.—(Lehigh Portland Cement Co., Allentown, Pa.). Determination of the differences in amount of shrinkage and expansion of cement bars 2 x 2 x 10 cm., under the same conditions of exposure. Special type gage equipped with an Ames dial graduated to hundredths of a millimeter, and which can be read to thousandths of a millimeter, is used for measuring volume changes. Attention is given to volume changes in different cements in relation to free lime content, strength, behavior under accelerated curing, etc. This is a long-time investigation, and will be a source of information for some years.

Effect of Various Types of Gypsum on the Properties of Cement.—(Lehigh Portland Cement Co., Allentown, Pa.). Cement clinker was ground in a laboratory mill with different percentages of three types of gypsum: one normal, one containing a considerable quantity of anhydrite, and one by-product gypsum of the phosphate industry. The effects of each on the time of setting and tensile strength of standard sand mortar, and on compressive strength of concrete are being determined.

Portland Cements in Mortars and Concretes.—(Pennsylvania Department of Highways, Harrisburg). A study of the relation between the strength of portland cement, as revealed by results of standard laboratory tests, and the strength of the resulting concrete. The tests indicate that the strength of the concrete, particularly at early ages, is proportional to the strength of the cement used.

An Investigation of Lumnite Cement.—(Washington University, St. Louis). Compression tests of 1 : 2 : 4 Lumnite cement concrete of different water-cement ratios at 7 ages ranging from 1 day to 1 year. The conclusions from available data may be summarized as follows:

(1) Variation of water-cement ratio with lumnite cement had an effect similar to the variation with portland cement; that is, a high water content decreased the strength.

(2) Curing after 24 hours apparently had no effect on strength except in the case of immersed specimens, the strength of which was lower than that of the air-cured specimens.

Strength of Cement Mortar and Concrete.—(Iowa State Highway Commission, Ames). Laboratory and field tests to determine the relationship between the strength of cement mortar and concrete.

Effect of Quality of Portland Cement upon Strength of Concrete.—(Michigan State Highway Lab., Ann Arbor, and U. S. Bureau of Public Roads). A study of the relationship between tensile strength of cement and strength of the resulting concrete.

Relation Between Tensile Strength of Cement and Strength of Result-

ing Concrete.—(Minnesota Highway Department, St. Paul). Parallel tests on cements of different strength and on concretes made from these cements.

Humidity Equilibria of Hardened Cement Pastes.—(Research Laboratory, Portland Cement Association, Chicago). Many of the properties of concrete are dependent upon the degree of combination of water with the cement and the tenacity with which the water is held by the cement under various conditions to which the concrete is subjected. It is known in a qualitative way that the water which goes into a state of stable combination with cement is affected by various factors, but few data of a quantitative nature are available. These tests are being made to obtain some quantitative measure of the effect on the quantity of water held in chemical combination or by capillary attraction of the following factors: (1) quantity of mixing water, (2) age of the cement paste, (3) temperature of curing, (4) temperature of testing, and (5) moisture condition of air.

II. AGGREGATES.*

Mineralogical Content of Maine Sands in Relation to Mortar Strength.—(University of Maine, Orono). This investigation is an attempt to ascertain the effect of mineralogical character of Maine sands on resulting tensile strength of 1: 3 mortar briquets at 7 and 28 days. Increase of granitic material (portion of sand which is either quartz, feldspar, or mica) results in lowering of mortar strength. For a complete report of these tests, see *Proceedings*, National Academy of Sciences, Vol. 13, p. 351, June, 1927.

Mechanical Analysis of Sand.—(University of Maine, Orono). Statistical study of laboratory sampling, and inter-relationship of size of sand particles in Maine sands. Results published in Bulletin 18 of the Maine Tech. Exp. Station.

Influence of Iron Content on Mortar Strength.—(University of Maine, Orono). Data indicates that iron content of sand has important influence on tensile strength of mortar, the larger the amount of iron the greater the average tensile strength. Report published in *Proceedings*, National Academy of Sciences, Vol. 13, p. 263, April, 1927.

Use of Louisiana Gravels as Aggregates.—(Louisiana State University and Louisiana Highway Commission, Baton Rouge). Compression, flexure, and wear tests being made to determine methods of proportioning for specification basis.

Effect of Character of Coarse Aggregate on Concrete.—(Minnesota Highway Department, St. Paul). Transverse and compression tests of concrete using various gravels and crushed stones. Results show that physical characteristics of aggregates have some effect on strength of concrete, especially in transverse tests. Sandstone weaker in cross bending than the compressive strength would indicate.

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*See also addenda, p. 776.

Application of Water-Cement Theory to Testing of Sand.—(Minnesota Highway Department, St. Paul). Investigation to establish more reliable test for quality of concrete sands. Tensile, compressive, and transverse tests on fluid mortars in comparison with standard sand tests.

Relation between Standard Abrasion Tests and Abrasion Tests on Smaller Sizes of Gravel and on Crushed Gravels.—(Minnesota Highway Department, St. Paul). Abrasion tests on gravel of sizes recommended by committee on Materials of American Association of State Highway Officials for purpose of establishing specification limits for abrasion tests run on gravels smaller in size than present standard, and on gravels containing crushed material.

Soundness Tests on Gravel.—(Minnesota Highway Department, St. Paul). Proposed investigation to determine resistance to weather of various Minnesota gravels.

Concrete from Unsound Stone.—(Kentucky State Highway Department and University of Kentucky, Lexington). Stone used was a low-grade blue shale which failed under sodium sulphate test within 24 hours, and completely failed when exposed to weather within 30 days. Concrete made from this stone both as fine and coarse aggregate. Mortar briquets showed no sign of failure when given the sodium sulphate test, and retained their full strength. The concrete cylinders after six months' exposure to the weather, showed no sign of failure.

Use of Coarsely Graded Screenings with Sand that Does not Meet Specifications.—(Kentucky State Highway Department and University of Kentucky, Lexington). Mixing 1/3 part screenings with 2/3 parts sand increased mortar strength ratio from 0.5 to 1.1, using these two materials as fine aggregate in a properly designed mix produced satisfactory concrete strength.

Tests on Aggregates.—(Research Laboratory, Portland Cement Association, Chicago). Many questions have been raised regarding the various properties of the aggregate on the properties of the concrete, but very few conclusive experimental data are available on many of these questions. This investigation was undertaken with a view to clearing up some of the present uncertainties, the purpose being to study as fully as possible the rôle of the aggregate in the finished concrete. The tests include studies on the relation between shape, size, gradation, surface peculiarities, durability, and other properties of aggregates on the workability and strength of the resulting concrete.

Study of Methods of Determining Moisture in Sand.—(Research Laboratory, Portland Cement Association, Chicago). A study of methods of determining the moisture content of sands. In addition to drying, the following five methods were used: (1) displacement with cylindrical container; (2) displacement with Chapman flask; (3) use of McIlvaine moisture meter; (4) Collins' hydrometer method; and (5) drying to constant weight by igniting mixture of sand and alcohol.

III. PLAIN CONCRETE.

The researches under way on plain concrete which are described below have been listed under the following classifications:

Admixtures	Poisson's ratio
Autogenous healing	Proportions
Carbon dioxide	Retempering
Curing	Sea water
Dams	Strength:
Durability	Compressive
Field tests and control	Flexural
Flow under load	Tensile
Jigging and vibration	Temperature effects
Mixing water	Test methods
Pavements	Volume changes
Permeability	Workability
	Yield

ADMIXTURES.*

Effect of Hydrated Lime on Concrete.—(University of Missouri, Columbia). Tests conducted during construction of the University of Missouri stadium on concrete with and without hydrated lime. Results given in *Concrete*, vol. 31, p. 16, July, 1927.

Atomite as an Admixture in Concrete.—(Engineering Experiment Station, Oregon Agricultural College, Corvallis). A study of the consistency, strength, permeability and wear resistance of concrete made with an admixture of atomite, a pulverized diatomaceous earth mined in Oregon.

Co-operative Studies to Determine the Fineness of Powders Too Fine to Be Separated by Means of Sieves.—(Lehigh Portland Cement Company, Allentown, Pa., in co-operation with sub-committee on Size and Shape of A. S. T. M. Committee E-1). Development of means of classification and designation of various particle sizes by specific measurements so that standard methods of mechcanical analysis may be extended to very fine powders. Analyses were made of various powders using the air-classifier; microscopical measurements will be made and results compared with computed sizes.

Effect of Diatomaceous Earth on Strength and Other Properties of Concrete.—(University of California, Berkeley). Tests made with Celite during 1925 were repeated using pacatome, another brand of diatomaceous earth, in concretes of various cement ratios, various fineness moduli, and of constant consistency. Compressive strength, modulus of elasticity, and yield were studied.

Admixtures in Concrete.—(Minnesota Highway Department, St. Paul). Transverse tests were made on concretes containing admixtures of calcium chloride, calatom, and hydrated lime for comparison with tests made on plain portland cement concrete and lumnite cement concrete, first using

*See also addenda, p. 776.

the same water-cement ratio, and then using the same flow. As shown by the flow and slump, calcium chloride slightly increased workability, while the other admixtures decreased it. There were no marked advantages in using any of these admixtures.

Tests of Portland Cement Colors.—(Research Laboratory, Portland Cement Association, Chicago). Continuing investigation of tinting quality and color permanence of 275 commercial colors in mortars upon exposure to weather. While the character of the aggregate has but little effect on the color of the mortar at early ages, it is evident from the data on hand that neutral colored aggregates or aggregates of colors harmonizing with the pigment used are necessary if a color, which will be permanently pleasing, is to be obtained. The pigments themselves have shown very little fading in most cases, but exposure of the aggregate has caused a change in the appearance of the mortar. Strength tests have shown that with the exception of carbon black, the pigments, which are otherwise suitable, do not have any conspicuously adverse effects on the strength of portland cement mortar when added in quantities commonly used.

AUTOGENOUS HEALING.

Autogenous Healing of Portland Cement Mortar Briquets.—(University of Colorado, Boulder). Distinct healing occurs following original tension tests at ages of 3, 7, and 28 days, 3 and 6 months. Greatest healing follows test at early age; will reheal after retest to less extent. In general, rich mixtures heal more than lean ones. All water-cement ratios heal. Healing takes place in both running and stagnant water, but no definite quantitative laws are in evidence yet.

CARBON DIOXIDE.

Resistance of Concrete to Water Acidified with Carbon Dioxide.—(Washington State Highway Department, Olympia). Tests on 500 2 x 4-in. cylinders to determine rate and extent of attack under hydrogen-ion concentrations ranging from pH 5.8 to pH 7.0, and factors influencing resistance. During first month of exposure at pH 5.8, average loss of lime from specimens was 7 per cent of original quantity present in cement. Rate of loss decreased to about .75 per cent per month at end of 6 months. At pH 7.0, change in weight of specimens indicates very small loss of lime.

CURING.*

Various Methods of Curing Concrete.—(Ohio State Highway Department, Columbus). Effect of wet earth, wet straw, calcium chloride (both on surface and in the mix), and sodium silicate curing on compressive and flexural strength of concrete.

Effect of Moist Closet Temperature on Strength of Standard Cement Mortar Briquets.—(Indiana State Highway Comm., Indianapolis). Tests designed to show importance of rigid control of moist closet temperature.

*See also addenda, p. 776.

Calcium Chloride as Curative Agent.—(Pennsylvania Department of Highways, Harrisburg). Investigation was begun in 1918 and has been completed. Standard tests on mortars and concrete, and service tests on concrete in which calcium chloride was used integrally show that calcium chloride is an efficient curing agent, and recognition has been given it in specifications as an alternative method of curing.

Field Curing of Concrete.—(Minnesota Highway Department, St. Paul). Determination of relative value of different methods of curing concrete in field. Specimens used were beams sealed with asphalt on sides, ends, and bottoms, only the top being subjected to curing. The methods used were wet earth, calcium chloride, sodium silicate, wet burlap, and no curing.

DAMS.

Physical Properties of Concrete.—(University of California, Berkeley). Study of the properties of concrete which have a bearing on interpretation of data collected in the field in connection with the Stevenson Creek arch dam investigation sponsored by Engineering Foundation covers data on temperature and humidity control, modulus of elasticity, volumetric changes, effect of fines upon shrinkage, flow under continued stress, permeability, and coefficient of thermal expansion.

Investigations of Models of Arch Dams.—(University of Colorado, Boulder, U. S. Bureau of Reclamation, and Engineering Foundation). Purpose of investigation is to increase knowledge of arch dam analysis. The first model to be tested is one of the Stevenson Creek test dam. It will be about one-tenth actual size, will be loaded with mercury, and accurate strain and deflection measurements will be taken.

DURABILITY.

Durability of Concrete.—(Hydro-Electric Power Commission, Toronto). A continuing investigation begun in 1924 covering a wide variety of concretes and mortars. Thus far studies in absorption by capillary action have been made on mortars and concretes, besides freezing and thawing tests on small and large specimens, and microscopic studies of pore distribution in concretes.

Methods of Rendering Concrete Resistant to Alkali Attack.—(Iowa State Colleges, Ames). Studies of methods of making concrete resistant to alkali attack, especially in the field of concrete drain tile. Small concrete cylinders immersed in various alkali solutions are observed from time to time, and deterioration and change in volume noted. No data available at present, but preliminary report in preparation.

FIELD TESTS AND CONTROL.

Proportioning of Materials.—(Wisconsin State Highway Commission, Madison). Beams and cylinders made on two paving jobs, and cores taken from road in an investigation of the strength of concrete in the field, using weight and volumetric proportioning of the aggregates.

Transverse Tests of Concrete in Field.—(Pennsylvania Department of Highways, Harrisburg). Principal factors in favor of transverse test for determining quality of concrete are its cheapness, simple operation, reliability, and concordance of results. Actual tests conducted on each contract in the state show increased strength due to closer control of consistency. The opening of roads to traffic is thus placed on a sound engineering basis, as this is done when the required strength is obtained rather than at some stated period.

Field Tranverse Tests of Concrete.—(Minnesota Highway Department, St. Paul). Transverse tests made on 6 x 6 x 30-in. beams using several methods of loading. Standardization of this test is important as tests show that careful attention must be given to method of applying load, to method of supporting beam, and to the rate of application of load.

FLOW UNDER LOAD.

Effect of Proportions of Materials on Plastic Flow of Concrete.—(Ohio State University, Columbus). A continuing investigation which includes effect of properties of materials on plastic flow of concrete. Conclusions drawn from available data are that plastic flow is very much greater at the early ages of loading and for the rich concrete.

Flow of Concrete under Continuous Compressive Strength.—(University of California, Berkeley). A continuing investigation, the results of which appear to establish that:

(1) The older the concrete at time of loading, the less rapid the rate of flow.

(2) The flow varies with magnitude of the stress, but this variation is not uniform, being more rapid at higher stresses.

(3) After long period of time, flow is materially less for water-soaked specimens than for those cured in air, though in some instances the rate of flow is more rapid for wet specimens than for dry during the period immediately following application of load.

(4) Six months after loading, wet specimens show a flow which is in general less than the instantaneous deformation which accompanies the application of load, but dry specimens show a flow materially greater than the strain accompanying application of load.

(5) In a general way flow varies with the instantaneous deformation and hence varies inversely as the modulus of elasticity of the concrete.

JIGGING AND VIBRATION.

Effect of Jigging during Setting Period on Strength of Concrete.—(University of California, Berkeley). Method was described in last year's report of this committee. Results to date are as follows:

(1) Jigging of 6 x 12-in. concrete cylinders increases compressive strength from 70 per cent to more than 100 per cent of strength of unjigged concrete for a 1: 6 mix tested at 28 days. Gain in strength when tested at 28 days, however, appears to bear no relation to gain in strength at 90 days.

(2) Rate of jigging has no effect upon final strength, but slower rates require a longer time of jigging to produce same increase in strength.

(3) Rapid jigging produces segregation which, if carried on too long, develops smaller increase in strength over unjigged concrete than that developed by shorter periods of rapid jigging.

(4) Celite in small quantities (2 per cent by weight of cement) prevents segregation of jigged concrete, but strength is less than for concrete without celite. With 10 per cent of celite, the jigged concrete showed an increase in strength of 117 per cent over the unjigged concrete with celite, and gave a greater strength than the jigged concrete without celite.

(5) Clay in small quantities (1.6 per cent. to 5 per cent by weight of cement) prevents segregation of jigged concrete, but strength is less than for concrete without clay.

MIXING WATER.

Effect of Removal of Excess Mixing Water upon Strength of Concrete. —(Washington University, St. Louis). Study of the effect of removal of water on strength of concrete which is poured too wet, and of effect of absorbent molds. Concrete of approximately a 1: 2: 4 mix was poured into absorbent molds of Rockwood, a gypsum product, using water-cement ratios of 1.2, 1.5, and 1.8. The molds were usually stripped at 6 or 7 days, and the majority of tests made at age of 7 days. Comparison was made of all mixes with standard 6 x 12-in. cylinders made and cured in steel molds. Results thus far obtained show that removal of excess mixing water by use of absorbent molds produces concrete which at 7 days has a strength greater than the strength at 28 days predicted from Abrams' curve. The percentage of increase is greater as the water-cement ratio is increased.

PAVEMENTS.

Pavement Condition Survey.—(Michigan State Highway Department, Lansing). Correlation of pavement conditions with soil, under drainage design, pavement design, age, and traffic.

PERMEABILITY.*

Permeability Tests.—(University of Washington, Seattle). An investigation to determine rate of flow of water through concrete block 8 in. thick under pressure of 150 lb. per sq. in. for various water-cement ratios, for various slumps with constant water-cement ratio, for various percentages of sand and gravel, and various gravels. Effect of pressure also studied. Tests thus far made show that no water flows through block

*See also addenda, p. 776.

with a water-ratio 0.90; slight leakage with a water-ratio 1.00; and sharp rate of increase of flow for water-cement ratios greater than 1.00.

Permeability Tests.—(Detroit Edison Co., Detroit). Investigations being made for purpose of establishing danger line in cement economy for general foundation concrete. Six x 12-in. test cylinders, same materials but with varying cement content, were placed under head of water and loss of head recorded after 7 and 28 days curing. Cylinders then broken and studied. The consistent results of the tests already completed indicate the possibility of definite conclusions when sufficient number of tests has been made.

Waterproofing Compounds and the Permeability of Concrete.—(University of California, Berkeley). Tests begun in 1926 to determine relative efficiency of various integral waterproofing compounds. Flat, disc-like specimens under pressure to top surface and collected in funnel clamped to lower surface. Preliminary tests show that 2-in. discs of well-graded 1:2:4 concrete were watertight under heads of 400 ft. Distilled water used, and that which percolated through specimens subjected to chemical analysis. Specimens tested at various ages.

POISSON'S RATIO.

Modulus of Elasticity and Poisson's Ratio for High Alumina Cement Concrete.—(University of Minnesota Engineering Experiment Station, Minneapolis). Martens type extensometers used in measurement of lateral and longitudinal deformations of cylindrical concrete specimens of varying characteristics.

Modulus of Elasticity and Poisson's Ratio.—(University of California, Berkeley). Tests begun in 1924, and will be completed in April, 1928, when 200 6 x 12-in. cylinders will have been tested in compression. The only variables are richness of mix and age of concrete. Both axial and transverse strains measured with mirror extensometers, observations being made to millionths of an inch. Results so far obtained may be summarized as follows:

(1) The older the concrete, the more nearly do axial and transverse strain diagrams approach straight lines.

(2) Increase in cement ratio is accompanied by increase in modulus of elasticity until certain maximum is reached beyond which, if cement ratio is still further increased, there is reduction in value of modulus of elasticity.

(3) Modulus of elasticity increases with age, but after one year increase is very slow. At age of two years, modulus of elasticity may be 50 per cent higher than at one month.

(4) In a general way, Poisson's ratio varies with modulus of elasticity, increasing with age of concrete and with cement ratio until the maximum is reached, beyond which an increase in cement ratio results in a decrease in Poisson's ratio. However, this is not so marked as the case of modulus of elasticity.

(5) Poisson's ratio varied with stress, being somewhat greater for low unit stresses than for high ones. In general, for unit stresses above 200 lb. per sq. in., the ratio does not vary more than 10 per cent.

Determination of the Modulus of Elasticity of High Alumina Cement Concretes.—(University of California, Berkeley). Mortar strength tests were made on 2 x 4-in. cylinders, and on tension briquets. Modulus of elasticity of 6 x 12-in. cylinders determined by use of mirror extensometer. Strength tests also made.

PROPORTIONS.

Economics of Concrete Mixtures.—(Hydro-Electric Power Commission, Toronto). Development of data whereby probable best proportions for any given set of aggregates can be closely approximated in advance, and effect of cost of screening, regrading, or other changes in aggregates can be determined so that their suitability may be judged. Same data, because of their completeness, will be available for many other studies of concrete mixtures.

Proportioning of Concrete by Weight.—(Pennsylvania Department of Highways, Harrisburg). Results of this investigation, which was carried on to insure greater uniformity in concrete and to compensate for bulking of sand, showed less variation in strength of concrete proportions by weight than by volume and greater yield for weight proportioning.

Design of Mix Using Low Grade Aggregates.—(Kentucky State Highway Department and University of Kentucky, Lexington). About 80 per cent of the deposits of inferior sand and gravel investigated gave satisfactory strengths with cement factor increased from 5 to 25 per cent, depending on the aggregate.

Design of Concrete of High Early Strength Using Standard Aggregates, Ordinary Portland Cement, and Calcium Chloride.—(Kentucky State Highway Department and University of Kentucky, Lexington). By this method it was possible to construct a slab bridge and at the same time keep the road open to traffic. Cylinders were tested at 1, 2, 3, 4, 7 and 28 days; temperature range was from 25 to 70 deg. F. At 4 days it was found that concrete had sufficient strength, and the bridge was opened. The mix required 2.25 bbl. cement per cu. yd. with 3 per cent calcium chloride. If bridge had been constructed during the summer, it could have been opened in 3 days with safety.

RETEMPERING.

Effect of Retempering on Strength of Concrete.—(University of Kansas, Lawrence). Investigation to ascertain how long centrally-mixed concrete may be held before placing.

Tests of Retempered Concrete.—(Research Laboratory, Portland Cement Association, Chicago). A continuing investigation, the purpose of which is to determine the effect of retempering and prehydration, both on

the workability of concrete and on the resulting strength. Some of the more significant features covered by the investigation were studies of:

(1) Strengths and workabilities of concrete mixes of a wide range of water-cement ratios after standing in air-tight cans for various periods from 0 to 6 hours after mixing.

(2) The amount of water required to restore concrete to its original condition of workability (as measured by the flow table) after standing 2 to 6 hours subsequent to mixing, and its effect on the strength.

(3) Effect of premixing cement and water for periods up to 30 min. before mixing with aggregate for ½ to 10 min.

Workability tests were made on concrete, using a 30-in. flow table. Compression specimens were 6 x 12-in. cylinders.

SEA WATER.

Resistance of Concrete to Sea Water.—(U. S. Bureau of Yards and Docks, Washington, D. C.). Tests in progress since 1925 to determine best mix, aggregates, admixtures, surface treatment, and cement for concrete exposed to sea water.

Effect of Sea Water on Strength of Plain Concrete.—(University of California, Berkeley). In this investigation the effect of sea water for mixing and curing on the compressive strength of 6 x 12-in. cylinders was studied. The results were as follows:

(1) A decided decrease in strength was observed when sea water was used for mixing.

(2) Sea water, when used for curing, has no effect on fresh water concrete.

STRENGTH: COMPRESSIVE.

Age-Strength Relations for Concrete.—(Lehigh Portland Cement Co., Allentown, Pa.). An investigation of factors influencing the age-strength curve. Aside from characteristics of the cement, such factors may be type of aggregates, proportions, consistency, water-cement ratio, and absolute strength. These investigations have eliminated type of aggregate as a major factor, varying proportions have not been included in these studies, and the last three factors are now under investigation. The indications are that either wetter consistencies or higher water ratios retard the gain in strength at early ages, so that the familiar age-strength curve may be concave downward for relatively stiff mixtures, approximately a straight line for medium consistencies, and concave upward for wet mixtures.

Studies of Unit Compressive Strength of Concrete.—(Lehigh Portland Cement Co., Allentown, Pa.). These studies made on concrete from aggregate ¾ in. in maximum size molded in 3 x 6-in. and 6 x 12-in. cylinders. At 28 days the average strength of the 3 x 6-in. cylinders was 2.2 per cent less than that of the 6 x 12-in.; at earlier ages the agreement was even closer. The mean deviation from the average for sets of ten 6 x 12-in.

cylinders was less than 3 per cent at all ages of test, and that for sets of fifty 3 x 6-in. cylinders was less than 4 per cent at all ages of test.

Effect of Curing Condition and Condition at Test Upon Compressive Strength.—(University of Colorado, Boulder). The purpose of this series of tests is to furnish data to previously published findings (See *Proceedings*, Am. Soc. C. E., Jan., 1927; *Proceedings*, A. S. T. M., 1927; *Proceedings*, A. C. I., 1926).

Relation Between Strength and Elasticity of Concrete in Tension and Compression.—(Iowa State College, Ames). Cylinders of various proportions of materials at various stages and cured by different methods were tested to obtain data on behavior of concrete in compression or tension.

STRENGTH: FLEXURAL.*

Effect of Repeated Stresses on Strength of Concrete.—(University of Texas Engineering Experiment Station, Austin). Materials are being assembled for this study of the effect of various factors on the ultimate flexural strength of concrete under repeated load. The apparatus to be used is unique in that four specimens can be tested simultaneously.

STRENGTH: TENSILE.

Tension, Compression, and Transverse Tests of Plain Concrete.—(Research Laboratory, Portland Cement Association, Chicago). A continuing investigation described in the previous report of this committee. The tests were designed to show the relation between the tensile, flexural, and compressive strength of plain concrete, and between the modulus of elasticity in tension and compression as affected by size, grading, and type of aggregate, quantity of cement and mixing water, age, curing condition, and other factors.

TEMPERATURE EFFECTS.

Effect of Temperature at Time of Test on Strength of Concrete.—(University of California, Berkeley). Tests of 6 x 12-in. cylinders or various cement ratios and of 1: 3 mortar briquets and 2 x 4-in. cylinders at different ages. Specimens tested at 130 deg. F. and at 10 deg. F.

Effect of Alternate Freezing and Thawing on Strength of Concrete.—(University of California, Berkeley). Tests at intervals of 7 days on compression and tension specimens cured at normal temperatures and alternated between temperatures of 10 deg. F. and 110 deg. F. and tested.

Length of Time to Protect Concrete in Cold Weather.—(Minnesota Highway Department, St. Paul). Concrete beams were exposed to freezing at different ages, and some repeatedly frozen and thawed. Beams were cured indoors at approximately 40 deg. and 70 deg. F. It was found that exposure immediately after casting is most injurious. Concrete cast and cured at 70 deg. F. is about one-third stronger than that cast and cured at 40 deg. F. Concrete is not harmed by frost after three days' protection, but needs additional curing to develop its full strength.

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*See also addenda, p. 776.

TEST METHODS.

Effect of End Conditions on Strains in Concrete Cylinders.—(University of California, Berkeley). Investigation to determine effect upon observed axial and lateral deformations of 6 x 12-in. cylinders when loads are not applied uniformly over top and bottom of specimen. All test specimens will be identical. One-third will have their ends ground smooth and will be loaded over the entire top and bottom, one-third will be axially loaded through a 3-in. plate at top and bottom, and the remainder will be axially loaded through annular rings 1 in. wide and 6 in. outside diameter, placed at both ends of the specimen.

Value of Sand Bearings in Eliminating Capping of Cylinders.—(New Hampshire Highway Testing Laboratory, Concord). Tests to determine the value of sand bearings in eliminating capping of cylinders especially those made in the field. Details of methods used are given in New Hampshire *Highways*, August, 1926.

A Study of the Uniformity of Concrete.—(Research Laboratory, Portland Cement Association, Chicago). This investigation was made to determine the uniformity of test results obtained with present methods of making and testing specimens, and to determine the advisability of modifying them in certain details. Compression specimens of 6 x 12-in. cylinders were made for test at ages of 7 days to 1 year from concrete in which the mix and consistency were varied over a wide range. Comparisons were made of the strengths obtained when a graphite grease was used on the top and bottom plates in place of paraffined paper. The effect of retaining all the mixing water in the mold was studied by clamping the molds to the base plates and sealing all joints with paraffin. Specimens of identical mixtures were made on the same day and on different days so that a determination of the day-to-day and batch-to-batch variations could be studied.

VOLUME CHANGE.

Expansion and Contraction of Concrete Due to Changes in Moisture Conditions.—(University of California, Berkeley). About 300 3 x 3 x 40-in. bars with steel contact points in the ends ranging from 1 month to 4 years old are under observation. Some specimens are continuously in water, some are continuously in dry air, and others are alternately wet and dried. The following are some of the results:

(1) Oven-dried specimens stored in air of constant humidity and constant temperature expand and absorb moisture rapidly at first, and then at a gradually decreasing rate until finally a state of volumetric weight and equilibrium is reached. Their rate of increase in length and weight varies with the degree of humidity, and the magnitude of their expansion and absorption is a function of the humidity.

(2) Specimens stored in water increase in both length and weight, rapidly at first, and then at a decreasing rate.

(3) Green and water-soaked specimens stored in dry air both shrink and lose weight, the rates of shrinking and water loss becoming less with passage of time.

(4) When concrete is thoroughly dried, it takes on a permanent shrinkage which is not recovered when it is again saturated, though it returns practically to its original water-soaked weight.

(5) Oven-dried concretes varying as to richness of mix, stored in air at a given humidity, have an expansion and absorption varying directly as the richness of the mix.

(6) When water-soaked concretes varying as to richness of mix are stored in dry air, the shrinkage is greater for rich mixes than for lean ones, and is also likely to be greater where the aggregates are of high surface modulus than where they are of low surface modulus.

(7) With granitic concretes containing varying percentages of granitic dust (fine material passing the No. 100 sieve), the percentage of fines has no appreciable effect upon the magnitude of the shrinkage when the concrete is dried, nor upon the expansion when the concrete is water-soaked. However, this is not true for other than granitic aggregates [see (6) above].

Study of Volume Changes, Their Causes and Control.—(Research Laboratory, Portland Cement Association, Chicago). Various factors mentioned in the literature on the subject are being investigated, among which are the following:

(1) Effect of regaging and remixing portland cement at various periods,

(2) Effect of sealing specimens of portland cement to prevent evaporation during the hardening period,

(3) Variation in size of the particles of cement,

(4) Study of other cementing materials of various combinations,

(5) Behavior of inert powders mixed with water,

(6) Variation in size and shape of the specimens,

(7) Study of consolidation with various liquids differing in surface tension,

(8) Study of cements stored under various conditions for periods up to five years,

(9) Effect of variations in storage conditions of specimens themselves, such as curing in moist air, in ordinary air, and under water for various periods.

Thermal Expansion of Concrete.—(University of California, Berkeley). It has been found that, for a 1:5 concrete made of well-graded granitic aggregates containing a slight excess of fines, there was very little, if any, difference between the thermal coefficient of expansion at high temperatures and that at low temperatures. It was also found that wet specimens have

a slightly higher coefficient than dry specimens. The results are summarized in the following table:

Condition	Coefficient of Thermal Expansion per 1 deg. F. At 7 Weeks	At 4 Months
Wet	0.00000447	0.00000410
Dry	0.00000379	0.00000390

Other specimens which are now curing are made of concretes of various mixes.

Expansion and Contraction of High Alumina Cement Concretes Due to Changes in Moisture Conditions.—(University of California, Berkeley). This investigation is being carried on using the equipment and following the procedure outlined above for portland cement concretes.

A Study of Moisture-Volumetric Changes in Plain Concretes.—(Stanford University, Stanford University, Cal.). Observations are being made on changes in volume during first 48 hours, effect of admixtures on volume changes, and internal stress caused by moisture-volumetric changes. Measurement of changes in length are being made during curing and during immersion in water after being oven-dried. Stress-strain relation is measured when specimens are dry and saturated, and under repeated stress.

WORKABILITY.

Study of Methods of Determining Workability of Concrete by Use of the Slump Cone and Flow Table.—(Research Laboratory, Portland Cement Association, Chicago). The purpose of this study was to determine whether slump and flow tests as now made are being used to the best advantage or whether a combination of the two tests could be used to better advantage in determining the workability of concrete. The tests covered a wide range of mixes using three consistencies for each mix. In the flow table tests, the number of ½-in. drops ranged from 0 to 25 using a 6¾ x 10 x 5-in. cone on a 30-in. flow table. Repeat tests are made on concrete molded in a 4 x 8 x 12-in. slump cone.

YIELD.

Cement Factor.—(Kentucky State Highway Department and University of Kentucky, Lexington). A study of the factors which influence the quantity of cement in a given mix. The cement factor cannot be controlled when aggregates are measured under ordinary field conditions; that is, loose and damp. Under such conditions the cement may show a so-called "over-run" of as much as 20 per cent.

IV. RESEARCHES ON REINFORCED CONCRETE.

Current researches in the field of reinforced concrete are as follows:

ARCHES.

Analysis of Rubber Models of Skew Arches.—(South Dakota State School of Mines, Rapid City). A comparison of the results of tests on rubber models under vertical loads with those obtained by theoretical analysis.

CHIMNEYS.

Determination of Stress in Reinforced-Concrete Chimneys.—(University of Toronto, Toronto). Results of this investigation were published in November, 1927, in pamphlet on "Charts for Approximate Stress Determination in Reinforced-Concrete Chimneys to Facilitate Work of Chimney Design." This study consisted mainly in charting basic formulas.

CULVERT PIPE.

Strength of Culvert Pipe and Determination of Loads Caused by Embankments.—(Iowa State College, Ames). Determination of theory of loads on culverts, and ordinary supporting strengths of various types of culverts. For results of this investigation, which has been in progress for several years, see Bulletins 76, 79 and 80 of Iowa Engineering Experiment Station.

FRAMES.

Tests of Reinforced-Concrete Frames.—(University of Illinois, Urbana). Study of resistance of frames to static and repeated lateral loads, and vibration of frames under impact. Horizontal loads were applied to frame by means of jack and dynamometer, repeated loads by use of fatigue machine. Vibrations were measured by means of mirror extensometers from which beams of light were thrown on photographic film.

SLABS.

Distribution of Shearing Stresses in Concrete Slabs under Concentrated Loads.—(Iowa State College, Ames). Investigation to determine the effective width of a portion of a floor slab which may be designed as a beam to carry concentrated loads. Special study is being given to shearing stresses. The shearing stresses at the support of the test slabs are assumed to be a function of the end reaction, the distribution of this end reaction being measured when a concentrated load is placed at various points on the slab. This distribution is being measured by a series of levers, ten in all, which are arranged to act as one abutment of the slab. When a concentrated load is applied, a counterweight is moved on each lever until all are balanced, the position of the counterweight indicating the amount of the reaction which is being supported by each lever.

V. SUGGESTED RESEARCHES ON CONCRETE AND RELATED SUBJECTS.

The following list of subjects suggested for research was compiled from replies to the committee's questionnaire:

AGGREGATES.

Study to determine accuracy and reliability of two methods recently proposed for rapid determination of moisture content of sand: (a) salt solution method, (b) water addition method.

Fine and coarse aggregates in concrete as a field for needed research.

Method of determining quality of coarse aggregate.

Use of high early strength cements for acceptance tests of fine and coarse aggregates.

Abrasion tests for fine and crushed gravel.

Effect of surface condition of coarse aggregates on strength of concrete as revealed by compressive and transverse tests.

PLAIN CONCRETE.

Thorough investigation of admixtures in concrete with relation to effect on strength, density, workability, watertightness, etc.: Determination of the proportions of hardened concrete; curing procedure for high alumina cement concrete; study of various methods of curing pavements; effect on surface of various methods of curing pavements; effect of temperature during curing; effect of oven drying on strength of moist-cured concrete; thorough study of effect of temperature on rate of hardening; effect of freezing on strength of concretes of varying moisture contents; durability of concrete under various conditions of exposure; durability of concrete from high early strength cements; most advantageous methods of finishing roads; properties of light-weight concretes; effect of time of mixing on retempered concrete of various slumps and water-cement ratios; planes of weakness in slabs and joints; economic possibilities of using concretes of higher designed strengths than 2,000 lb. per sq. in. at 28 days; investigation of strength at different heights of concrete members placed vertically; design of paving slabs on strength basis; effect on early transverse strength of methods employed to give high early compressive strength; effect of proportions of fine and coarse aggregate and consistency on strength of concrete as revealed by compressive and transverse tests; determination of relation of transverse to compressive strength for various types of beam-testing apparatus; standardization of the transverse test for field specimens; expansion and contraction of concrete block under variable temperature and moisture conditions; expansion and contraction of masonry mortars; development of a test for determining workability of concrete; and quantitative investigation to determine fundamental factors which control slump and workability of concrete.

REINFORCED CONCRETE.

Anchorage of footing steel; hooks for anchorage of bars, radius of bend, compression under the hook, tendency of hook to straighten, comparison with straight bar; test of skew arches under horizontal load; possibility of using elastic gelatin models to investigate probable stress distribution in certain highly indeterminate structures such as arch dams; comparison of measured stresses with theoretical stresses on upstream and downstream face of an arch dam; restraining effect of lateral ties in columns; effect of concrete shrinkage on column stresses; and study of column splices.

Experimental investigation to secure data for design of reinforced-concrete wing-walls. Should include a study of external pressure acting on wing-walls, as well as a study of distribution of internal stresses within the wall.

Durability of out-door structures, especially with reference to corrosion of steel and resulting failure.

Analytical investigations, with some experimental verifications of:

(1) Flat-slab theory and practical applications;
(2) Multiple-arch bridges and dams;
(3) Monumental building frame;
(4) Economic possibilities of rib compensation in arches:
(5) Local stresses in reinforced concrete at re-entrant angles, openings, and at other points of discontinuity of loading or shape;
(6) Deflections of beams and slabs with special reference to determination of moments of inertia and stiffness coefficients respectively, for use in analytical problems;
(7) Combination of structural steel shapes and built-up members with concrete;
(8) Investigation by means of models of stability of long-span slender arches;
(9) Mechanical methods of analysis for statically indeterminate structures.

VI. REFERENCES TO PAPERS AND REPORTS ON RESEARCHES PUBLISHED DURING 1927.

In compiling the accompanying list of references the aim has been to report only the more noteworthy articles. The references were compiled principally from the following publications:

American Concrete Institute *Proceedings;* American Society Civil Engineers *Proceedings;* American Society Testing Materials *Proceedings;* Ceramic Abstracts; Chemical Abstracts; *Concrete; Engineering News-Record;* Engineering Experiment Bulletins; Highway Research Board *Proceedings;* Highway Research *News; Industrial and Engineering Chemistry; Journal,* American Chemical Society; National Bureau of Standards, Washington, D. C.: circulars, scientific papers, technical papers; *Public Roads; Rock Products;* Foreign Technical Publications: *Annales des Ponts et Chaussées,* Paris; *Beton und Eisen,* Berlin; British Board of Fire Underwriters' Reports, London; British Institute of Structural Engineers, London; *Canadian Engineer; Chemiker Zeitung,* Gothen; *Comptes Rendus,* Paris; *Concrete and Construction Engineering,* London; *Deutscher Ausschuss fur Eisenbeton,* Berlin; *Der Bauingenieur,* Berlin; Engineering Abstracts of Institution of Civil Engineers, London; *Engineering Journal,* (Canada); Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Berlin; Forscherarbeiten auf dem Gebiete des Eisenbetons, Berlin; *Moniteur Scientifique,* Paris; Scientific and Industrial Research Technical Papers, London; *Stahl und Eisen,* Dusseldorf; *Tonindustrie Zeitung,* Berlin; *Zeitschrift der Deutscher Ing.,* Berlin; and *Zement,* Berlin.

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Zement, v. 15, p. 512, 1926.

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Journal Phys. Chemistry, v. 31, p. 607, April, 1927.
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ADDENDA.

AGGREGATES.

Concrete-Making Ability of Different Natural Sands, (Iowa State Highway Commission, Ames).
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Use of Soft Limestone in Concrete, (Iowa State Highway Commission, Ames).
A study of the effect upon strength and durability of concrete made with a relatively soft limestone as coarse aggregate.

ADMIXTURES.

Use of Calcium Chloride in Concrete, (Iowa State Highway Commission, Ames).
Specimens were made in the field and in the laboratory in order to determine the effect of calcium chloride as an admixture in concrete.

Use of Admixtures in Concrete, (Iowa State Highway Commission, Ames).
A study of the effect of various inert admixtures in concrete.

CURING.

Methods of Curing Concrete, (Iowa State Highway Commission, Ames).
Study of the effect of various methods of curing concrete pavements.

PERMEABILITY.

Waterproofing Concrete, (Iowa State Highway Commission, Ames).
Study of the efficacy of various methods of waterproofing concrete by surface treatments. Specimens coated or painted, immersed in water, removed and weighed at stated intervals.

STRENGTH: FLEXURAL.

Fatigue Tests of Concrete, (Iowa State Highway Commission, Ames).
Determination of the effect of repeated stresses on the strength of concrete.

Report of Committee E-5, Aggregates.

The work of this committee has progressed along the lines laid out last year in studies of:

(1) A thorough revision of the Tentative Purchase Specifications for Concrete Aggregates adopted by the Institute in 1926, still incomplete as this report is submitted.

(2) Method of Test for Abrasion of Gravel. The present report includes a discussion submitted by the sub-committee on gravel, Stanton Walker, chairman, concerning the development of the proposed abrasion test, and a proposed tentative method of test for abrasion of gravel.

This report has been submitted to letter ballot of the committee, which consists of 15 members, of whom 9 have voted affirmatively, none negatively, and 6 have refrained from voting.

R. W. Crum, *Chairman.*

Requirements of Gravel as an Aggregate for Concrete.

Reported by Sub-Committee on Gravel of Committee E-5.

The sub-committee on gravel of Committee E-5 presented a report[1] before the 1927 convention which discussed, in a general way, requirements of gravel as an aggregate for concrete. The principal purpose which this report served was to point out the inadequacy of our information concerning the effect of the various characteristics which govern the concrete-making properties of gravel. Brief discussions and summaries of information were given under the following heads:

Hardness and strength of gravel, durability of gravel, cleanness of gravel, grading of gravel, resistance to high temperatures, recommended studies, and factors to be considered in specifications for gravel.

In the discussion under "recommended studies" it was stated, among other recommendations, that "further studies of the hardness and strength of gravel particles should be made, and it is suggested that modifications of the standard Deval test and the crushing test are worthy of further attention." It is the purpose of this year's report of the sub-committee to present a further discussion of the modifications of the Deval abrasion test suitable for use in testing gravel.

Abrasion Tests of Gravel.

Engineers have given considerable attention to the relation between the resistance to abrasion of aggregate and its concrete-making properties, but very little conclusive information has been developed. In fact, existing data would seem to indicate that the resistance to abrasion of gravel, as measured by any of the tests thus far developed, has little bearing upon its concrete-making properties.

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[1] 1927 *Proceedings,* American Concrete Institute, p. 574.

Development of Abrasion Test.—The standard Deval abrasion test for crushed stone was developed during the latter part of the nineteenth century in connection with early macadam road construction in France.[2] The practice of using this test soon found its way to the United States and it became one of the standard test methods of the American Society for Testing Materials in 1908.[3] Under the title, "Standard Method of Test for Abrasion of Road Material," it has been continued as a standard of that society since that time without change. While the "standard" test is not adapted for use in connection with studies of concrete aggregates, and was not developed with that purpose in view, it was carried over into concrete specifications with the development of highway construction and is now generally used as one of the measures of quality of crushed stone for that purpose.

It was at once evident that the standard test could not be applied to gravel because of the requirements for shape and size of particles composing the prepared test sample. As a result of this, a number of different investigations were carried out, looking toward the development of a suitable modification of the test. In these investigations the attempt was made not only to develop a test suitable for gravel, but also to establish a relationship between the resistance to abrasion of gravel as measured by the various modified tests and that of crushed stone as measured by the standard test.

Among the more prominent of the early investigators were A. S. Rea[4] of the Ohio State Highway Department, H. H. Scofield[5] of Cornell University, then at Purdue University, F. H. Jackson[6] of the U. S. Bureau of Public Roads, H. S. Mattimore[7] at that time with the New York State Highway Department and later of the Pennsylvania State Highway Department, and Duff A. Abrams,[8] then with the Structural Materials Research Laboratory and now of the International Cement Corporation.

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[2] "Original work of Deval in Developing Abrasion Test for Crushed Rock for Use in Macadam Road Construction," *Annales des Ponts et Chaussées*, 1879, Bulletin, Ministere des Travaux Publics, 1881.

[3] "Standard Abrasion Tests for Road Materials, *Proceedings*, A. S. T. M., Vol. 8, 1908; describes Deval abrasion test for crushed stone.

[4] "Standard Abrasion Test for Gravel Employed by the Ohio State Highway Department," by A. S. Rea, *Good Roads*, June 6, 1914.

[5] "Abrasion Test for Gravel Aggregate," by A. S. Rea, *Concrete Highway Magazine*, June, 1918; describes modified form of Deval abrasion test. See also "Abrasion Tests for Stone, Gravel and Similar Aggregate," by H. H. Scofield (Discussion by H. S. Mattimore), *Proceedings*, A. S. T. M., Vol. 18, 1918; *Good Roads*, Vol. 16, p. 108, Sept. 21, 1918; Abstract, *Engineering News-Record*, Vol. 81, p. 53, July 4, 1918. Deval rattler designed which allows dust to escape.

[6] "Standard Deval Abrasion for Rocks," by F. H. Jackson, *Proceedings*, A. S. T. M., Part II, 1920; *Public Roads*, July, 1920; discusses causes for error and methods of avoiding them.

[7] "Use of Slotted Cylinder for Deval Abrasion Test of Rocks," report, N. Y. Commissioner of Highways, 1917. See also "Report of State Commissioner of Highways, state of New York, 1917; results of tests on sandstone, limestone, slag, and gravel, using standard cylinder and slotted cylinder.

[8] "Selection of Mineral Aggregate for Concrete Roads," by D. A. Abrams, *Proceedings*, A. R. B. A., 1922, p. 126; *Concrete*, Vol. 20, p. 159, April, 1922; Abstract, *Good Roads*, Vol. 62, p. 67, Feb. 1, 1922; Abstract, *Engineering and Contracting*, Vol. 57, p. 419, May 3, 1922; Abstract, *Canadian Engineer*, Vol. 43, p. 211, Aug. 1, 1922. Abrasion tests in Deval machine by four methods and corresponding strength and wear of concrete. See also discussion by Duff A. Abrams of paper on "An Impact Test for Gravel, by F. H. Jackson, *Proceedings*, A. S. T. M., Part II, 1922, p. 370.

These investigations covered studies of the effect of size and grading of sample, amount of abrasive charge, revolutions of cylinder, effect of removing dust through slots in the cylinder, tests of type of abrasion machine other than the Deval machine, and other variables. This report does not review these early tests in detail. It is sufficient to say that, as a result of these investigations, the American Association of State Highway Officials tentatively adopted in 1917 the method of test first proposed by Mr. Rea in 1914.[9] This test consists of running a sample of 5,000 grams, made up of 2,500 grams of ½-in. to 1-in. material and 2,500 grams of 1-in. to 2-in. material, in a standard Deval abrasion testing machine for 10,000 revolutions, with six 1⅞-in. cast iron balls added as an abrasive charge.[10] In 1920 the specification for the grading of the samples was revised to require 1,250 grams of each of the following four sizes: ½ in. to ¾ in.; ¾ in. to 1 in.; 1 in. to 1½ in.; and 1½ in. to 2 in.[11]

This latter test method has been incorporated in most specifications for gravel of the various state highway departments; limiting percentages of wear have generally been placed at about 15 per cent, although in a few cases as low as 10 per cent and as high as 25 per cent is specified. These limiting percentages reflect the judgment of the engineer, since no conclusive information has been developed which shows a relation between the results of this test and that of the standard test for stone, or which shows any dependence of the concrete-making properties of gravel on its resistance to abrasion measured in this way.[12]

Objections to Present Test Method.—The tentative test method described above is open to several objections, among the most important of which may be stated the following:

1. Due to the grading of sample required, the test cannot be made on all aggregates since the necessary sizes may not be available.

2. In the case of gravel containing a considerable proportion of sizes finer than ½ in., the test sample is not representative of the grading as a whole.

3. Where the sample contains a considerable quantity of crushed particles the results obtained by this test are not comparable on the same basis as for samples containing no crushed particles.

Recognizing these objections and realizing also the desirability of obtaining more definite information on which to base specification limits, various agencies have carried out further studies of the present modification of the test method and have also investigated other modifications.

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[9] Loc. Cit.

[10] "Standard Forms for Specifications, Tests, Reports, and Methods of Sampling for Road Materials," United States Department of Agriculture Bulletin No. 555, as recommended by the First Conference of State Highway Testing Engineers and Chemists, Washington, D. C., Feb. 12 to 17, 1917.

[11] "Tentative Standard Methods of Sampling and Testing Highway Materials," U. S. Dept. of Agriculture Department Bulletin No. 1216, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with federal-aid road construction.

[12] See *Proceedings*, American Concrete Institute, 1927, Report of Committee E-5 on Aggregates, Appendix 1 and Appendix 2, pp. 585 and 587.

Recent Investigations.—This sub-committee is endeavoring to collect and correlate available information obtained from the more recent investigations, the information from which has not yet been made available by publication. With this in view, and in connection with the work of Sub-Committee V of Committee C-9 of the American Society for Testing Materials, a considerable amount of correspondence has been carried out and much valuable information has been submitted. It is not proposed to present a comprehensive study of these data, nor would the limits of space allowed to this report permit it. In what follows a brief statement is given describing the nature of the tests which have been carried out.

Michigan.—A thesis, submitted by Bernard T. Schad to the University of Michigan for the Degree of Master of Science in Engineering, entitled "Investigation of Abrasion Tests of Road Gravel," contains a great deal of valuable information. The thesis, which is quite comprehensive in scope, gives a review of the development of abrasion tests and proposes a modification in the existing tentative standard for grading of sample. A considerable number of tests were made in connection with the investigation and the following variables were studied:

1. Grading of Sample.
    a. 1,250 grams of each of the following sizes: ¼ to ¾ in., ¾ to 1 in., 1 to 1½ in., and 1½ to 2 in. (same as the tentative method of the American Association of State Highway Officials).
    b. 1,666.6 grams of each of the following sizes: ¼ to ½ in., ½ to ¾ in., ¾ to 1 in.
    c. 2,500 grams of each of the following sizes: ½ to ¾ in., and ¾ to 1 in.
2. Effect of Abrasive Charge.
    a. No abrasive charge.
    b. Six 1⅞ in. cast-iron balls.
3. Tests with Plain and Slotted Cylinders.
4. Tests of 38 different samples of gravel, using a plain cylinder and the grading described under 1-b above. The cementing value of the abraded material was also determined.

The conclusions drawn by Mr. Schad as a result of these tests may be summarized as follows:

The test using the grading consisting of equal parts of three sizes of material, ranging from ¼ to 1 in. in size, seemed to adapt itself to gravel used for gravel roads. It was considered evident, from the tests, that the inclusion of the material finer than ½ in. and coarser than ¼ in. was amply justified, due to the important effect which this size had on the results of the wear test. The standard cylinder was considered to be more satisfactory than the slotted one because by retaining the abraded dust particles the test more nearly approached the conditions found in the road.

The addition of the abrasive charge of six 1⅞-in. cast-iron spheres was considered to be an advantage because the tests made in this way detected excessive amounts of soft material.

Illinois.—A comprehensive investigation of abrasion tests of gravel has been carried out at the highway laboratory of the Illinois State Highway Department. These data have been made available through the courtesy of A. E. Stoddard, assistant engineer of materials. The tests include studies of the effect of grading of sample, use of the plain and slotted cylinder, and shape of aggregate particle. They also include comparisons between the standard test for crushed stone and the modified test for gravel. It is not practical to summarize the results of these tests in this report. Mr. Stoddard plans to make them available in the form of a paper to be offered to the American Society for Testing Materials.

California.—A letter from E. T. Maddocks, testing engineer of the California Division of Highways, states the following:

"We have been making the shot abrasion tests for gravel for several years and have found the tests to be very satisfactory. The main objection to the test in our minds is that it does not test the soft particles from ½ in. down. While it is true that, in many cases, we receive samples which do not contain the larger sizes required, we can usually obtain the proper sizes by going to the pit and securing same.

"We have been experimenting with the abrasion test and have developed what we call the disintegrated granite abrasion test. The rock for this test includes the small sizes only, 50 per cent being between the ¾-in. and ½-in. screen and 50 per cent between the ½-in. and the No. 3 screen. The total charge of rock used is 4,200 grams. While we have made no close study of the relationship existing between the regular shot abrasion and the disintegrated granite test, it would appear that the results obtained with a total charge of rock of 4,200 grams are very close to the regular shot tests. This, of course, is in cases where the character of the rock is about the same from the large to the small sizes.

"We have made one departure from the standard test in that we use 1⅞-in. steel ball bearings instead of the cast-iron balls specified."

Ohio.—Recent investigations carried out by A. S. Rea of the Ohio State Highway Department have shown the desirability of making provisions for carrying out the Deval test on aggregate samples which do not contain particles coarser than 1 in. The necessity of taking into account the shape of particle was also recognized, and a formula developed for determining the permissible percentages of wear for different percentages of crushed particles.

The test is made in the standard Deval abrasion testing machine using 10,000 revolutions of the cylinder and six 1⅞-in. cast-iron balls as an abrasive charge. The grading of the sample for the test depends on the size of the gravel, as follows:

Gravel Graded from 1/4 in. to 2 in.

Passing 2-in. screen, retained on 1-in. screen	2,500 grams
(At least 1,250 grams shall be finer than the 1½-in. screen.)	
Passing 1-in. screen, retained on ¾-in. screen	1,250 "
Passing ¾-in. screen, retained on ½-in. screen	1,250 "
Total	5,000 "

Gravel Graded from 1/4 in. to 1 in.

Passing 1-in. screen, retained on ¾-in. screen	2,500 grams
Passing ¾-in. screen, retained on ½-in. screen	2,500 "
Total	5,000 "

The formula for taking into account the presence of crushed particles is the same as that described in the proposed "Method of Test for Abrasion of Gravel" given further on in this report.

Portland Cement Association.—In some of the early work of the Structural Materials Research Laboratory, carried out through the co-operation of the Portland Cement Association and the Lewis Institute, Chicago, methods of making abrasion tests were studied. As a result of these preliminary investigations a series of tests was carried out in 1920 in which the following variables were studied:

1. Effect of number of revolutions of cylinder. Tests were made for revolutions from 1,000 to 10,000. Five aggregates, different in type, and three gradings of sample were used.
2. Effect of weight of sample. The weight of sample was varied from 2,000 to 7,000 grams. The tests were made with three aggregates, different in type, and of the same grading.
3. Effect of number of shot added as an abrasive charge. The number of 1⅞-in. cast-iron balls were varied from 0 to 15. Five aggregates, different in type, and two gradings were used.
4. Effect of grading of sample. Tests were made using 57 different gradings, consisting of different proportions of three sizes of materials.
5. Effect of shape of particle. Tests were carried out for study of wear of aggregates of different types after crushed particles had been worn to rounded form.

These data are being prepared for presentation in the form of a paper which it is expected will be available this spring.

Proposed Method of Test for Abrasion of Gravel.—As a result of an informal conference held by the following: A. S. Rea, F. H. Jackson, Wallace F. Purrington, A. T. Goldbeck, and Stanton Walker, it was decided to submit the following proposed method of test, to the Committee on Materials of the American Association of State Highway Officials, Com-

mittee C-9 on Concrete and Concrete Aggregates and Committee D-4 on Road and Paving Materials of the American Society for Testing Materials, and Committee E-5 on Aggregates of the American Concrete Institute.

Representatives of the sub-committees of each of these organizations, having this problem in charge, are included in the men named above.

PROPOSED TENTATIVE METHOD OF TEST FOR ABRASION OF GRAVEL.

A. Test for gravel containing no crushed pieces.—(1) The sample shall consist entirely of uncrushed fragments of gravel and the test shall be made using one of the four gradings, (A, B, C or D), given in Paragraph (3). The grading most nearly representing that of the material furnished for the work shall be selected for the test.

(2) The aggregate shall first be screened into the different sizes required for the test and the material of these sizes shall be washed and dried.

(3) The sample shall consist of 5,000 grams of the dry gravel, with the different sizes combined to conform to one of the following four gradings:

Grading	Size of Sieves or Screens Retained on	(Square or Circular Openings)[15] Passing	Per Cent
A	½	¾	25
	¾	1	25
	1	1½	25
	1½	2	25
B	½	¾	25
	¾	1	25
	1	1½	50
C	½	¾	50
	¾	1	50
D	¼	½	50
	½	¾	50

(4) The sample for the test shall be placed in the cast-iron cylinder of the Deval abrasion testing machine as specified for the standard abrasion test for stone.[13] Six cast-iron spheres 1.875 in. in diameter and weighing approximately 0.95 lb. (0.45 kilo.) each shall be placed in the cylinder as an abrasive charge.[14]

(5) The duration of the test and the rate of rotation shall be the same as specified for the standard test for stone, namely, 10,000 revolutions

[13] See A. S. T. M. Standard Method of Test for Abrasion of Road Material (Serial Designation D 2-08).

[14] These spheres are the same as those used in the standard rattler test for paving brick.

[15] As originally proposed the method required the use of circular openings. The question whether circular or square openings shall be used for testing aggregates is now being studied by the Sectional Committee on coarse sizes of aggregates, Committee E-1 of A. S. T. M.

at a rate of 30 to 33 revolutions per minute. At the completion of the test the material shall be taken out and screened over a No. 12 mesh sieve, conforming to the requirements of the Standard Specifications for Sieves for Testing Purposes of the American Society for Testing Materials (Serial Designation: E 11-26). The material retained upon the sieve shall be washed, dried and weighed. The difference between this weight and the weight of the original sample, expressed as a per cent of the original weight, shall be considered as the loss by abrasion.

(6) When the gravel has a specific gravity below 2.20, a sample of 4,000 grams, instead of 5,000 grams, shall be used for the test. The testing procedure shall be the same in all other respects.

(7) When the gravel, as used in the work, contains as much as 15 per cent of material finer than ½ in., but is of such size that either Grading A, B or C would be used for the abrasion test, a second abrasion test shall be made using Grading D, if, in the opinion of the engineer, the particles finer than ½ in. are not at least equal in hardness to those coarser than ½ in.

B. Test for gravel containing crushed pieces.—(8) Gravel containing more than about 10 per cent of crushed pieces shall, for the purpose of this test, be considered as crushed gravel. In such cases, the abrasion test shall be made on a representative sample of the whole, including the crushed pieces, following the procedure described in Paragraphs 1 to 7. The per cent, by weight, of crushed pieces shall be determined and the permissible per cent of wear which shall govern for any given sample shall be calculated from the following formula:

$$W = \frac{A\,L + (100 - A)\,L'}{100}$$

In which A = per cent of uncrushed pieces.

$100 - A$ = per cent of crushed pieces.

L = maximum per cent of wear permitted by the specifications for gravel containing no crushed pieces.

L' = maximum per cent of wear permitted by the specifications for gravel consisting entirely of crushed pieces.

W = permissible per cent of wear.

It should be recognized that the different gradings of sample will require the statement of different limiting percentages of wear. A number of organizations, notably the Bureau of Public Roads, the Ohio State Highway Department, and the New Hampshire State Highway Department, have volunteered to carry out investigations along these lines.

Future Work of Sub-Committee.—During the coming year the subcommittee expects to study the results of tests carried out on the different gradings of sample recommended above. It is hoped that by the time of the next annual meeting sufficient information will have been obtained to permit of making recommendations for limitation on the loss by abrasion.

Discussion—Committee E-5 Report.

D. D. McGuire.—I would like to ask Mr. Jackson if he has arrived at any specific limit for this method of test? Mr. McGuire.

F. N. Jackson.—No sir, we have not. The plan was to set up a method of test and then proceed with the examination of various materials with the idea of eventually setting up limits which would be applicable to various sections of the country for the various sizes. Mr. Jackson.

R. E. Roscoe.—I would like to say that the Engineers' Society of Pennsylvania, at Pittsburgh, has formulated a specification for aggregates in which they limit the amount of coarse in the sand and in the gravel. Our coarse aggregate in Pittsburgh is usually gravel. Mr. Roscoe.

Reinforced-Concrete Building Regulations and Specifications.

Submitted by Committee E-1.

The presentation of this report marks the completion of one of the most active and, perhaps, one of the most important year's efforts of the committee's recent history. The proposed "Standard Building Regulations for Reinforced Concrete," which it is submitting, is something more than just another committee report. These chapters on reinforced-concrete design and construction, prepared for use in a building code, are the result of the combined efforts of this committee and the committee on Engineering Practice of the Concrete Reinforcing Steel Institute.

At the last meeting, the American Concrete Institute adopted as a Tentative Standard the "Proposed Building Regulations for the Use of Reinforced Concrete," which had been before the membership for two years. By this action, there was placed before the public the second of two authoritative standards in suitable form for use as part of a city building code—the Institute standard and that of the Concrete Reinforcing Steel Institute.

These two standards, while in complete agreement on the essential principles of reinforced-concrete design, were somewhat at variance on many of the less important items and wholly different in form. Both of these standards were being urged for adoption in the building codes of a number of cities. At the time of the 1927 convention, Committee E-1 discussed this situation and the chairman was authorized to enter into preliminary negotiations with the committee on Engineering Practice of the Concrete Reinforcing Steel Institute to determine if there was any possibility that the two committees could unite on a common standard. The advantage of having a common standard, if one could be developed, appealed strongly to both committees. The preliminary meetings between representatives of the two committees indicated that agreement on such a standard was entirely possible.

These preliminary negotiations were the beginning of a very serious study of both codes by the two committees. This study has resulted in the agreement of the two committees on the proposed "Standard Building Regulations for the Use of Reinforced Concrete," submitted with this report. The preparation of this proposed standard has received extended consideration for a period of ten months, involving a number of sessions of both committees, meeting separately and jointly, and of many sessions of a smaller group representing both committees.

This proposed standard in the opinion of the committee is a decided improvement over the tentative standard adopted in 1927. The most important change is in the method of presentation. The text has been greatly simplified and clarified, notably in the chapters on shear, bond, and flat

slabs. Of the changes relating to materials and the basis of design, the following are the more important:

(1) The discrimination in the 1927 report against the use of rail-steel reinforcement has been removed.

(2) The unit tensile stresses in intermediate grade billet-steel reinforcing bars and in rail-steel reinforcing bars have been increased from 18,000 to 20,000 lb. per sq. in., while the stress in all web reinforcement has been decreased to 16,000 lb. per sq. in.

(3) The method of specifying concrete has been changed in detail, though not in essence, except that provision is made for the use of concrete with strength in excess of 3,000 lb. per sq. in. where very rigid control is assured.

(4) The maximum permissible shearing stress on concrete beams has been reduced from $0.12f'_c$ to $0.09f'_c$ with the provision that beams or girders may be used with shearing stresses greater than $0.09f'$ (but not greater than $0.12f'_c$) when such beams or girders are clearly indicated on the plans, and when the designer shall personally supervise the construction of such members.

The committee submits this report with the recommendation that it be adopted as a tentative standard.

This report has been submitted to a letter ballot of the committee consisting of 19 members of whom 18 have voted in the affirmative, none in the negative, and 1 has failed to vote.

F. R. McMILLAN, *Chairman.*

INTRODUCTION.

These regulations have been prepared for use as part of a general building code. When so used, it is necessary that the following definitions, which give the meaning of certain terms as used in the regulations, become a part of the code. They should appear either in a general chapter in the code relating to definitions or in a chapter by themselves preceding these regulations for the use of reinforced concrete:

DEFINITIONS.

Aggregate.—Inert material which is mixed with portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone, blast-furnace slag, or similar materials.

Anchorage.—The embedment in concrete of a portion of a reinforcement bar, either straight or with hooks, designed to prevent pulling out or slipping of the bar when subjected to stress. (The anchorage of tension reinforcement in beams includes only the embedded length beyond a point of contra-flexure or of zero moment.)

Blast-Furnace Slag.—The non-metallic product, consisting essentially of silicates and alumino-silicates of lime, which is developed simultaneously with iron in a blast furnace.

Column.—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital.—An enlargement of the upper end of a reinforced-concrete column designed and built to act as a unit with the column and flat slab.

Column Strip.—A portion of a flat slab panel one-half panel in width occupying the two quarter-panel areas outside of the middle strip. (See Middle Strip.)

Add also definition as follows:

Combination Column.—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column.—A column in which a concrete core enclosed by spiral reinforcement and further reinforced by longitudinal bars encases a structural steel or cast-iron column designed to carry a portion of the load.

Concrete.—A mixture of portland cement, fine aggregate, coarse aggregate and water. (See Mortar.)

Consistency.—A general term used to designate the relative plasticity of freshly mixed concrete or mortar.

Crushed Stone.—Bedded rock or boulders, which have been broken by mechanical means into fragments of varying shapes and sizes.

Dead-Load.—The weight of the permanent parts of the structure.

Deformed Bar.—Reinforcement bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling so as to firmly engage the surrounding concrete. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcing and with cross wires not smaller than No. 10 may be rated as a deformed bar.

Diagonal Band.—In a four-way flat slab system a group of bars covering a width approximately 0.4 the average span, symmetrical with respect to the diagonal running from corner to corner of the panel.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel of a flat slab.

Direct Band.—In a four-way flat slab system, a group of bars covering a width approximately 0.4 l_1, symmetrical with respect to the line of centers of supporting columns.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section which lies between the centroid of the tension reinforcement and the compression surface in a beam or slab, and having a width equal to the width of the rectangular beam or slab, or the effective width of the flange of a Tee beam.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between its direction and that for which the effectiveness of the reinforcement is to be determined.

Flat Slab.—A reinforced-concrete slab generally without beams or girders to transfer the loads to supporting members.

Footing.—A structural unit used to distribute wall or column loads to the foundation materials.

Gravel.—Rounded particles larger than sand grains resulting from the natural disintegration of rocks. (See Sand.)

Laitance.—Extremely fine material of little or no hardness which may collect on the surface of freshly deposited concrete or mortar, resulting from the use of excess mixing water, usually recognized by its relatively light color.

Live-Load.—Loads and forces other than the dead-load.

Middle Strip.—A portion of a flat slab panel one-half panel in width, symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered.

Mortar.—A mixture of portland cement, fine aggregate, and water. (See Concrete.)

Negative Bending Moment.—That moment which exists between a support of a slab or beam and the point of inflection on either side of the support.

Negative Reinforcement.—Reinforcement so placed as to take tensile stress due to negative bending moment.

Paneled Ceiling.—A paneled ceiling refers to a flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length.—The distance in either rectangular direction between centers of two columns of a panel.

Pedestal.—An upright compression member whose height does not exceed three times its least lateral dimension.

Pedestal Footing.—A column footing projecting less than one-half its depth from the faces of the column on all sides and having a depth not more than three times its least width.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Bending Moment.—That moment which exists at all other points in beams or slabs except where negative moment exists.

Positive Reinforcement.—Reinforcement so placed as to take tensile stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Sec. 1002.)

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete at that section.

Rectangular Direction.—A direction parallel to a side of the panel of a flat slab.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting forces.

Sand.—Small grains resulting from the natural disintegration of rocks. (See Gravel.)

Screen.—A metal plate with closely spaced circular perforations. (See Sieve.)

Sieve.—Woven wire cloth with square openings. (See Screen.)

Strut.—A compression member other than a column or pedestal.

Surface Water.—By the term "surface water" is meant all water carried by the aggregate except that held within the aggregate particles themselves by absorption.

Wall Beam.—A reinforced-concrete beam which extends from column to column along the outer edge of a wall panel.

Water-Cement Ratio.—By the water-cement ratio is meant the total quantity of water entering the mixture including the surface water carried by the aggregate, expressed in terms of the quantity of cement. The water-cement ratio shall be expressed in U. S. gallons per sack (94 lb.) of cement.

Tentative Building Regulations for Reinforced Concrete.*

CHAPTER 1.

General.

101: *Scope:*

(a) These regulations cover the use of reinforced concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of this material. In all matters pertaining to the design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102: *Permits and Drawings:*

(a) Drawings and typical details of all reinforced-concrete construction showing the sizes and position of all structural members, metal reinforcement, and the live-load used in the design shall be filed with the department as a permanent record before a permit to construct such work shall be issued. All calculations made may be required by the department to be submitted with the drawings.

103: *Special Systems of Reinforced Concrete:*

(a) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with these provisions or not covered by them, shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction." This Board shall be composed of competent engineers, architects and builders. The Board shall have the power to investigate the data so submitted and to formulate rulings governing the design and construction of such systems, which ruling shall be of the same force and effect as the provisions of this code. This Board is to be designated as provided elsewhere in the code.

CHAPTER 2.

Materials and Tests.

201: *Tests:*

(a) The tests called for in these regulations when ordered in accordance with the provisions of this chapter by the commissioner of buildings or his authorized representatives shall be arranged for by the owner or his representative. No responsibility for the expense of these tests shall attach to the department of buildings. Such tests shall be made in accordance with the standard method of test covering the particular material under consideration, of the American Society for Testing Materials in effect on the date of the adoption of these regulations, except as noted herein.

(b) All such tests shall be made by competent persons. The com-

* The report of Committee E-1, Reinforced-Concrete Building Design and Specifications, carrying proposed building regulations for reinforced concrete is here published as amended on the floor of the convention, Feb. 29, 1928. As amended this report was adopted as Tentative Building Regulations for Reinforced Concrete (E-1A-28T).

petency of the persons making the tests shall be judged by their training and experience. The commissioner of buildings may disapprove for just cause those whose records show technical incompetency. Copies of the results of all tests shall be kept on file in the office of the commissioner of buildings for a period of two years after the acceptance of the structure. Tests shall be made on any material entering into concrete or reinforced-concrete construction when there is reasonable doubt as to its suitability for the purpose.

(c) The commissioner of buildings or his authorized representative shall have the right to require reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality. Specimens for such tests shall be taken at the place where concrete is being deposited, and shall be taken, cured, and tested in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation: C 39-27) of the American Society for Testing Materials.

202: *Load Tests:*

(a) The commissioner of buildings or his authorized representative shall have the right to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it was intended. Such tests shall not be required to be made on any concrete construction which is less than 60 days old.

(b) In such tests, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made, or where lawful, a lower rating shall be established. The structure will be considered to have failed to pass the test if within twenty-four hours after the removal of the load the slabs or beams do not show a recovery of at least 75 per cent of the maximum deflection shown during the twenty-four hours while under load.

203: *Inspection:*

(a) All concrete work shall be inspected by the architect or engineer responsible for its design or by a competent representative responsible to the architect or the engineer. A record shall be kept of such inspection which shall cover the quality and quantity of concrete materials, including water, the mixing and placing of the concrete, and the placing of the reinforcing steel. The inspection record shall also include a complete record of the progress of the work and of the temperatures, when these fall below 40 deg. F., and of the protection given to the concrete while curing. These records shall be available for inspection by the commis-

sioner of buildings at all times during the progress of the work and shall be preserved for two (2) years after the acceptance of the structure.

204: *Portland Cement:*

(a) Portland cement shall conform to the "Standard Specifications and Tests for Portland Cement" (Serial Designation: C 9-26) of the American Society for Testing Materials.

205: *Concrete Aggregates:*

(a) Concrete aggregates shall consist of natural sands and gravels, crushed rock, crushed air-cooled blast-furnace slag, or other inert materials having clean, uncoated grains of strong and durable minerals. Aggregates containing soft, friable, thin, flaky, elongated, or laminated particles totaling more than 3 per cent, or containing shale in excess of 1½ per cent, or silt and crusher dust finer than the No. 100 standard sieve in excess of 2 per cent shall not be used. These percentages shall be based on the weight of the combined aggregate as used in the concrete. When all three groups of these deleterious materials are present in the aggregates, the combined amounts shall not exceed 5 per cent by weight of the combined aggregate.

(b) Aggregates shall not contain strong alkali or organic material which gives a color darker than the standard color when tested in accordance with the "Standard Method of Test for Organic Impurities in Sands for Concrete" (Serial Designation: C 40-27) of the American Society for Testing Materials.

(c) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars. By maximum size of aggregate is meant the clear space between the sides of the smallest square opening through which 95 per cent by weight of the material can be passed.

206: *Water:*

(a) Water used in mixing concrete shall be clean, and free from strong acids, alkalis, or organic materials.

207: *Metal Reinforcement:*

(a) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" of Intermediate Grade[1] (Serial Designation: A 15-14), or for "Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A 16-14) of the American Society for Testing Materials. The provision in these specifications for machining deformed bars before testing shall be eliminated.

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[1] This recommendation is in accordance with "Commercial Standard No. 1 (New Billet-Steel Concrete Reinforcing Bars)" of the U. S. Department of Commerce, which establishes the intermediate grade as the single standard for billet-steel reinforcement. Until such time as existing stocks of structural and hard-grade billet-steel reinforcement, meeting the requirements of A. S. T. M. Specification A 15-14 are exhausted, these grades may be used with the unit stresses specified in Sec. 307.

(b) Wire for concrete reinforcement shall conform to the requirements of the "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A 82-27) of the American Society for Testing Materials.

(c) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Buildings" (Serial Designation: A9-24) of the American Society for Testing Materials.

(d) Cast-iron sections for composite or combination columns shall conform to "Standard Specifications for Cast-Iron Pipe and Special Castings" (Serial Designation: A44-04) of the American Society for Testing Materials.

208: *Storage of Materials:*

(a) Cement and aggregates shall be stored at the work in a manner to prevent deterioration or the intrusion of foreign matter. Any material which has deteriorated or has been damaged shall be immediately and completely removed from the work.

CHAPTER 3.

CONCRETE QUALITY AND WORKING STRESSES.

301: *Concrete Quality:*

(a) The working stresses for the design of reinforced-concrete structures shall be based upon the minimum ultimate 28-day strength of the concrete to be used in the structure in accordance with the values given in Sec. 306. All plans submitted for approval or used on the work shall clearly show the strength of concrete for which all parts of the structure were designed. The strength of concrete shall be fixed in terms of the water-cement ratio in accordance with one of the following methods:

(1) By established results for average materials, as provided in Sec. 302.
(2) By specific test of materials for the structure, as provided in Sec. 303.

(b) By the water-cement ratio is meant the total quantity of water entering the mixture including the surface water carried by the aggregate, expressed in terms of the quantity of cement. The water-cement ratio shall be expressed in U. S. gallons per sack (94 lb.) of cement.

302: *Water-Cement Ratio for Average Materials:*

(a) Where no preliminary tests of the materials to be used are made, the water-cement ratios shall not exceed the values in the following table. The mixes shown in the table are approximate only, and may require adjustment to give proper workability.

ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Cement Ratio U. S. gallons per 94-lb. sack of cement	Approximate Mix Volume of Portland Cement to Sum of Separate Volumes of Fine and Coarse Aggregate as Measured Dry	Assumed Compressive Strength at 28 days in pounds per square inch
	Plastic Concrete	
8¼	1 : 7	1,500
7½	1 : 6	2,000
6¾	1 : 5¼	2,500
6	1 : 4½	3,000
	Moderately Wet Concrete	
8¼	1 : 6½	1,500
7½	1 : 5½	2,000
6¾	1 : 4¾	2,500
6	1 : 4	3,000

NOTE: In interpreting this table, surface water contained in the aggregate must be included as part of the mixing water in computing the water-cement ratio.

(b) During the progress of the work, a reasonable number of compression tests shall be made as may be required by the commissioner of buildings, but at least one specimen shall be tested for each 100 cu. yd. of concrete being placed. The tests shall be made in accordance with provisions of Sec. 304. Should the average 28-day strength fall below the minimum ultimate strength called for on the plans, the commissioner of buildings shall have the right to require a load test under the provisions of Sec. 202.

303: *Water-Cement Ratio by Tests of Materials:*

(a) Where the water-cement ratios for the various strengths of concrete are to be established by test, these tests shall be made in advance of the beginning of operations using the materials proposed and consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation C 39-27) of the American Society for Testing Materials, including the provisions for curing in a moist room at 70 deg. F. and testing wet. A curve representing the relation between the average 28-day strength of the concrete and water-cement ratio shall be established for a range of values including all of the strengths called for in the plans. The tests shall include at least four different water-cement ratios and at least four specimens for each water-cement ratio. The water-cement ratio to be used in the structure shall be that corresponding to a point on the curve established by these tests representing a strength of concrete 15 per cent higher than the minimum ultimate strength called for on the plans and satisfactory evidence shall be submitted to show that these water-cement ratios are not exceeded. No substitution shall be made in the materials being used on the work without additional tests in accordance, herewith, to show the new water-cement ratios to be used.

(b) During the progress of the work, a reasonable number of additional 28-day compression tests may be required by the Commissioner of Buildings, but at least one specimen shall be tested for each 50 cubic yards of concrete of any one strength, and not less than two specimens of each strength of concrete for any one day's operation. Such tests shall be made in accordance with the provisions of Section 304. Should the average strengths of the control cylinders shown by these tests for any portion of the structure fall below the minimum ultimate 28-day strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mix or the water-cement ratios for the remaining portion of the structure and to require load tests as specified in Section 202 on the portions of the building affected. Should the average strengths shown by the cylinders cured on the job and tested subsequent to 28 days fall below the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength.

304: *Field Tests of Concrete:*

(a) Field tests of concrete, when required, shall be made in accordance with the "Standard Method of Making Compression Tests of Concrete" (Serial Designation C 39-27) of the American Society for Testing Materials with the following exceptions:

(1) Two sets of samples of concrete for test specimens shall be taken as the concrete is being delivered at the point of deposit, care being taken to obtain a sample representative of the entire batch.

(2) One set designated as control cylinders shall be placed under moist curing conditions at approximately 70 deg. F. within 24 hours after molding and maintained therein until tested.

(3) The second set, designated as job cylinders, shall be kept as near to the point of sampling as possible and yet receive the same protection from the elements as is given to the portions of the structure being placed. Specimens shall be kept from injury while on the work. They shall be sent to the laboratory not more than 7 days prior to the time of test and while in the laboratory shall be kept in the ordinary air at a temperature of approximately 70 deg. F.

(b) All specimens and tests shall be made by a properly qualified person or testing laboratory, who shall furnish the commissioner of buildings with a report, certified in the presence of a notary public, showing the results of tests and stating that they were made in accordance with the provisions of this code.

305: *Concrete Proportions and Consistency:*

(a) The proportions of aggregates to cement for concrete of any water-cement ratio shall be such as to produce concrete that will work readily into the corners and angles of the form and around the reinforcement without excessive puddling or spading and without permitting the materials to segregate or free water to collect on the surface. The combined aggregate shall be of such composition of sizes that when separated by the No. 4 standard sieve, the weight retained on the sieve shall not

be less than one-third nor more than two-thirds of the total nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the faces and corners of the members shall show smooth and sound throughout.

(b) The methods of measuring concrete materials shall be such that the proportion of water to cement can be accurately controlled during the progress of the work and easily checked at any time by the commissioner of buildings or his authorized representative.

306: *Allowable Unit Stresses in Concrete:*

(a) The unit stresses in pounds per square inch on the concrete to be used in the design shall not exceed the following values, where f'_c equals the minimum ultimate strength at 28 days.

DESCRIPTION	Allowable Unit Stresses: For any Strength of Concrete as Fixed by Test in Accordance with Sec. 303 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Cement Ratio in Accordance with Sec. 302: $f'_c =$ 2000 lb. $n = 15$	$f'_c =$ 2500 lb. $n = 12$	$f'_c =$ 3000 lb. $n = 10$
Flexure: f_c.				
Extreme fiber stress in compression (f_c)	$0.40f'_c$	800	1000	1200
Extreme fiber stress in compression adjacent to supports of continuous or fixed beams or of rigid frames (f_c)	$0.45f'_c$	900	1125	1350
Shear: v.				
Beams with no web reinforcement and without special anchorage of longitudinal steel (v_c)	$0.02f'_c$	40	50	60
Beams with no web reinforcement, but with special anchorage of longitudinal steel (v_c)	$0.03f'_c$	60	75	90
Beams with properly designed web reinforcement, but without special anchorage of longitudinal steel (v)	$0.06f'_c$	120	150	180
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel (v)	$0.09f'_c$	180	225	270
For conditions determining the use of greater shear values see Sec. 903(e).				
Flat slabs at distance d from edge of column cap or drop panel (v_c)	$0.03f'_c$	60	75	90
Footings where longitudinal bars have no special anchorage (v_c)	$0.02f'_c$	40	50	60
Footings where longitudinal bars have special anchorage (v_c)	$0.03f'_c$	60	75	90
Bond: u.				
In beams and slabs and one-way footings:				
Plain bars (u)	$0.04f'_c$	80	100	120
Deformed bars (u)	$0.05f'_c$	100	125	150
In two-way footings:				
Plain bars (u)	$0.03f'_c$	60	75	90
Deformed bars (u)	$0.0375f'_c$	75	94	112
(Where special anchorage is provided (see Sec. 903), double these values in bond may be used.)				
Bearing: f_c.				
Where a concrete member has an area at least twice the area in bearing (f_c)	$0.25f'_c$	500	625	750
Axial Compression: f_c.				
In columns with lateral ties (f_c)	$0.225f'_c$	450	563	675
In columns with continuous spirals enclosing a circular core:[1]				
Ratio of longitudinal reinforcement $p = 0.01$	$300 + 0.14f'_c$	580	650	720
0.02	$300 + 0.18f'_c$	660	750	840
0.03	$300 + 0.22f'_c$	740	850	960
0.04	$300 + 0.26f'_c$	820	950	1080
0.05	$300 + 0.30f'_c$	900	1050	1200
0.06	$300 + 0.34f'_c$	980	1150	1320
(Spiral reinforcement not to be less than ¼ the longitudinal.)				

[1] Unit stress in spirally reinforced columns = $[300 + (0.10 + 4p)f'_c]$.

307: *Allowable Unit Stresses in Reinforcement:*

(a) The following unit stresses in reinforcing steel shall not be exceeded:

Tension:

Intermediate grade billet steel[1]............ (f_s) = 20,000 lb. per sq. in.
Rail steel bars.......................... (f_s) = 20,000 lb. per sq. in.
Web reinforcement .... (f_v) = 16,000 lb. per sq. in.
Structural steel shapes.................. (f_s) = 18,000 lb. per sq. in.
Other steel reinforcement 50 per cent of the yield point stress, but not to exceed.... (f_s) = 20,000 lb. per sq. in.

Compression:

Bars .... nf_c
Structural steel section in composite columns......15,000 lb. per sq. in.
Cast iron section in composite columns............ 9,000 lb. per sq. in.

See Section 1106 for stresses in structural steel and cast iron not encased in concrete.

Structural steel section in combination column, see Section 1107.

CHAPTER 4.

MIXING AND PLACING CONCRETE.

401: *Removal of Water from Excavation:*

(a) Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the commissioner of buildings. Any flow of water into the excavation shall be diverted through proper side drains to a sump, or be removed by other approved methods which will avoid washing the freshly deposited concrete. Water vent pipes and drains shall be filled by grouting or otherwise, after the concrete has thoroughly hardened.

402: *Cleaning Forms and Equipment:*

(a) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the places to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and clay or cement tile that will be in contact with concrete shall be well drenched (except in freezing weather). Reinforcement shall be thoroughly cleaned of ice or other coatings.

403: *Inspection:*

(a) Concrete shall not be placed until the forms and reinforcement have been inspected by the architect or engineer responsible for the design or his authorized representative.

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[1] Until existing stocks of structural and hard grades of billet-steel reinforcement are exhausted, these grades, if conforming to the provision of Sec. 207. may be used with the following unit stresses:

Structural Grade .... (f_s) = 18,000 lb. per sq. in.
Hard Grade .... (f_s) = 20,000 lb. per sq. in.

404: *Mixing:*

(a) The concrete shall be mixed until there is a uniform distribution of the materials and the mass is uniform in color and homogeneous. The mixer shall be of such type as to insure the maintaining of the correct proportions of the ingredients. The mixing shall continue for at least one minute after all the ingredients are in the mixer.

405: *Transporting:*

(a) Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which will prevent the separation or loss of the ingredients. It shall be deposited as nearly as practicable in its final position to avoid rehandling or flowing. Under no circumstances shall concrete that has partially hardened be deposited in the work.

(b) When concrete is conveyed by chuting, the plant shall be of such size and design as to insure a practically continuous flow in the chute. The slope of the chute shall be such as to allow the concrete to flow without separation of the ingredients. The delivery end of the chute shall be as close as possible to the point of deposit. When the operation is intermittent, the spout shall discharge into a hopper. The chute shall be thoroughly flushed with water before and after each run; the water used for this purpose shall be discharged outside the forms.

406: *Placing:*

(a) When concreting is once started, it shall be carried on as a continuous operation until the placing of the section or panel is completed. Where construction joints are necessary, they shall be made in accordance with Sec. 507.

(b) Concrete shall be thoroughly compacted by puddling with suitable tools during the operation of placing, and thoroughly worked around the reinforcement, around embedded fixtures, and into the corners of the forms.

(c) Where conditions make puddling difficult, or where the reinforcement is congested, batches of mortar containing the same proportion of cement to sand used in the concrete, shall first be deposited in the forms and the operation of filling with the regularly specified mix be carried on at such a rate that the mix is at all times plastic and flows readily into the spaces between the bars.

(d) A record shall be kept on the work of the time and date of placing the concrete in each portion of the structure. Such record shall be kept until the completion of the structure and shall be open to the inspection of the commissioner of buildings.

407: *Curing:*

(a) Exposed surfaces of concrete shall be kept moist for a period of at least 7 days after being deposited. In hot weather, exposed concrete shall be thoroughly wetted twice daily during the first week.

408: *Depositing in Cold Weather:*

(a) When depositing concrete at freezing or near freezing temperatures, the concrete shall have a temperature of at least 50 deg. F., but not more than 120 deg. F. The concrete shall be maintained at a temperature of at least 50 deg. F. for not less than 72 hours after placing or until the concrete has thoroughly hardened. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing. No frozen materials or materials containing ice shall be used. Manure shall not be applied directly to concrete when used for protection.

CHAPTER 5.

FORMS AND DETAILS OF CONSTRUCTION.

501: *Design of Forms:*

(a) Forms shall conform to the shape, lines, and dimensions of the member as called for on the plans. They shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape and insure safety to workmen and passersby. Temporary openings shall be provided where necessary, to facilitate cleaning and inspection immediately before depositing concrete.

502: *Removal of Forms:*

(a) The removal of forms shall be carried out in such a manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, removable floor forms, beams and girder sides, column and similar vertical forms may be removed within 24 hours, providing the concrete has hardened sufficiently that it is not injured. In no case shall the supporting forms be disturbed until the concrete has hardened sufficiently to permit their removal with safety. Shoring shall not be removed until the member has acquired sufficient strength to support safely its weight and the load upon it.

503: *Cleaning and Bending Reinforcement:*

(a) Metal reinforcement, before being placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Reinforcement shall be formed to the dimensions indicated on the plans. Cold bends shall be made around a pin having a diameter of four or more times the least dimension of the bar.

(b) Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends not shown on the plans shall not be used. Heating of reinforcement for bending will not be permitted.

504: *Placing Reinforcement:*

(a) Metal reinforcement shall be accurately placed and secured, and shall be supported by concrete or metal chairs or spacers, or metal hangers. The minimum center to center distance between parallel bars shall be

2½ times the diameter for round bars or 3 times the side dimension for square bars; if the ends of bars are anchored as specified in Sec. 903, the center to center spacing may be made equal to 2 diameters for round bars or to 2½ times the side dimension for square bars, but in no case shall the clear spacing between bars be less than 1 in., nor less than 1⅓ times the maximum size of the coarse aggregate. Bars at the upper face of any member shall be embedded a clear distance of not less than one diameter, nor less than 1 in.

505: *Splices and Offsets in Reinforcement:*

(a) In slabs, beams, and girders, splices of reinforcement shall not be made at points of maximum stress without the approval of the commissioner of buildings. Splices, where permitted, shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices, the bars shall be spaced at the minimum distance specified in Sec. 504.

(b) Splices in column bars shall provide a lap of 24 diameters for deformed bars and 30 diameters for plain bars.

(c) Where changes in the cross-section of a column occur, the longitudinal bars shall be sloped for the full length of the column or offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion from the axis of the column shall not be more than 1 in 6.

506: *Protective Covering of Concrete:*

(a) At those surfaces of footings and other principal structural members in which the concrete is deposited directly against the ground, metal reinforcement shall have a minimum covering of 3 in. of concrete. At other surfaces of concrete exposed to the ground or weather, metal reinforcement shall be protected by not less than 2 in. of concrete.

(b) In fire-resistive construction, metal reinforcement shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 1½ in. in beams, girders, and columns, provided coarse aggregate is used, which is free from disruptive action under high temperatures, as, for example, limestone or trap rock; when impracticable to obtain aggregate of this grade, the protective covering shall be ½ in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in. placed 1 in. from the finished surface. In similar structures where the fire hazard is limited, the metal reinforcement shall not be placed nearer the exposed surface than ¾ in. in slabs and walls, or 1 in. in beams, girders, and columns.

(c) Cement or gypsum plaster, ¾ in. or more in thickness (on metal lath weighing not less than 2½ lb. per sq. yd. when used vertically, nor less than 3 lb. per sq. yd. when used horizontally) may be substituted for a part of the protective covering of concrete, provided that only two-thirds of the thickness of the plaster be considered effective and the concrete protection shall in no case be reduced to less than ¾ in.

(d) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

507: *Construction Joints:*

(a) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a horizontal joint is to be made, any excess water and laitance shall be removed from the surface after concrete is deposited. Before depositing of concrete is resumed, the hardened surface shall be cleaned and roughened and all weak concrete removed.

(b) At least 2 hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(c) Construction joints in floors shall be located near the middle of spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

CHAPTER 6.

DESIGN—GENERAL CONSIDERATIONS.

601: *Assumptions:*

(a) The design of reinforced-concrete members under these specifications shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending involving the following assumptions:

(1) The steel takes all tensile stress,

(2) The ratio n of the modulus of elasticity of the steel to that of the concrete shall be taken as follows (applies also for compression members):

$$n = \frac{E_s}{1{,}000f'_c} = \frac{30{,}000}{f'_c}$$

602: *Notation:*

(a) The symbols and notation used in these regulations are defined as follows:

a = width of face of column or pedestal;

α = angle between inclined web bars and axis of beam;

A = total area of top of pedestal, pier, or footing;

A' = loaded area of pedestal, pier, or footing at the column base;

A_c = area of core of spirally-hooped column measured to the outside diameter of the spiral;

A_g = gross area of tied columns with lateral ties;

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Sec. 1005, 1008, 1009, and 1010.

A_v = total area of web reinforcement in tension within a distance of s (measured perpendicular to the direction of the web reinforcement bar), or the total area of all bars bent up in any one plane;

b = width of rectangular beam or width of flange of T-beam;

b' = thickness of web in beams of I or T sections;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;

c = diameter in feet of column capital of a flat slab at the underside of the slab, or dropped panel. No portion of the column capital shall be considered for structural purposes which lies outside of the largest 90° cone that can be included within the outlines of the column capital;

c = projection of footing from face of column or pedestal;

d = depth from compression surface of beam or slab to center of longitudinal tensile reinforcement;

E_c = modulus of elasticity of concrete in compression;

E_s = modulus of elasticity of steel in tension or compression = 30,000,000 lb. per sq. in.;

f_c = compressive unit stress in extreme fiber of concrete in flexure or axial compression in concrete in columns;

f'_c = ultimate compressive strength of concrete at age of 28 days;

f_r = compressive unit stress in metal core;

f_s = tensile unit stress in longitudinal reinforcement;

f_v = tensile unit stress in web reinforcement;

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

l = span length of beam or slab (generally distance from center to center of supports; for special cases, see Sec. 702 and 1005);

l = span length of flat slab panel (usually expressed in feet) center to center of columns, in the direction in which moments are considered (see Sec. 1003);

l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;

M = bending moment or moment of resistance in general;

M_o = sum of positive and negative bending moments at the principal design sections of a panel of a flat slab (see Sec. 1003);

n = E_s/E_c = ratio of modulus of elasticity of steel to that of concrete;

Σo = sum of perimeters of bars in one set;

p = ratio of effective area of tensile reinforcement to effective area of concrete in beams = A_s/bd; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;

p_a = permissible unit stress on pedestal, pier, or footing when the full area is loaded;
P = total safe axial load on column whose length does not exceed 11 times its least cross-sectional dimension;
P' = total safe axial load on long column;
r_a = permissible unit working stress in concrete over the loaded area of a pedestal, pier, or footing;
R = least radius of gyration of a section;
s = spacing of stirrups measured perpendicular to the direction of the stirrup;
t = thickness of flange of T-beam;
t_1 = thickness of flat slab without dropped panels; or the thickness of flat slabs, including dropped panels where one is used;
t_2 = thickness of flat slab with dropped panels at points away from the dropped panel;
u = bond stress per unit of area of surface of bar;
v = shearing unit stress;
v_c = unit shearing stress permitted on the concrete of the web; the value depending on the anchorage of the longitudinal reinforcement;
V = total shear;
V' = excess of the total shear over that permitted on the concrete;
w = uniformly distributed load per unit of length of beam or slab;
w = upward reaction per unit of area of base of footing;
w' = uniformly distributed dead and live load per unit of area of a floor or roof (in flat slabs usually expressed in pounds per square foot);
W = total dead and live load uniformly distributed over a single panel area (in flat slabs usually expressed in pounds and includes the dead weight of any raised or depressed portions).

603: *Design Loads:*

(a) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

604: *Wind Loads:*

(a) Provisions shall be made for wind loads in accordance with the general provisions of the code of which this forms a part. In designing the members to resist wind loads, the allowable unit stresses for dead- and live-load and wind loads may be increased to 150 per cent of the allowable values specified in Sec. 306 and 307, but the section shall not be less than that required if the wind load be neglected.

CHAPTER 7.

FLEXURAL COMPUTATIONS AND MOMENT COEFFICIENTS.

701: *Formulas for Flexure:*

(a) Computations of flexural resistance of reinforced-concrete beams and slabs shall be based on the assumptions of Sec. 601. The customary formulas or their equivalent shall be used.

702: *Span Length:*

(a) The span length of freely supported beams and slabs shall be the clear span plus the depth of beam or slab, but shall not exceed the distance between centers of the supports.

(b) The span length for continuous or restrained beams built to act integrally with supports shall be the clear distance between faces of supports.

(c) For continuous or restrained beams having brackets built to act integrally with both beam and support and of a width not less than the width of the beam and making an angle of 45 deg. or more with the horizontal, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third more than the depth of the beam. No portion of such a bracket shall be considered as adding to the effective depth of the beam. Brackets making an angle of less than 45 deg. with the horizontal may be considered as increasing the effective depth of the beam, but not as decreasing the span length.

(d) Maximum negative moments are to be considered as existing at the ends of the span, as defined above.

703: *Depth of Beam or Slab:*

(a) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the top surface of the structural slab. Any floor finish not placed monolithic with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithic with the structural slab in buildings of the warehouse or industrial class where the finish is subjected to unusual wear from trucking or other causes, there shall be placed an additional depth of ½ in. over that used in the design of the member.

704: *Point of Inflection:*

(a) For the purpose of these regulations, the point of inflection in beams and slabs of equal spans symmetrically loaded shall be assumed to be located at the fifth point of the span as defined in Sec. 702.

705: *Distance between Lateral Supports:*

(a) The clear distance between lateral supports of a beam shall not exceed 32 times the least width of compression flange.

706: *Requirements for T-Beams:*

(a) In T-beam construction the slab shall be built integrally with the beam. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam,

and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(b) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(c) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a rib in ribbed floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, or in any case 18 in.

(d) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Sec. 504, relating to the spacing of bars, and 406(c), relating to the placing of concrete shall be fully met. In no case shall the area of steel in compression at any cross-section adjacent to the support exceed 2 per cent of the cross-sectional area of the stem of the beam in that section.

(e) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(f) Isolated beams in which the T-form is used only for purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

707: *Ribbed Floor Construction:*

(a) Ribbed floor construction includes floor systems of ribs and slabs placed monolithically in which the ribs are not farther apart than 36 in. face to face. The ribs shall be straight, not less than 4 in. wide, nor of a depth more than 3 times the width.

(b) Where removable forms or fillers not complying with (c) are used the thickness of the concrete slab shall not be less than 1/12 of the clear distance between ribs and in no case less than 2 in.

(c) When burned clay or cement tile are used and concrete is placed on the top of such tile, it shall not be less than 1½ in. in thickness, nor less than one-twelfth of the clear distance between ribs. When the tile are so placed that the joints in alternate rows are staggered, the webs of the tile in contact with the ribs may be included in calculations involving shear or negative bending moment. No other portion of the tile may be included in design calculations.

(d) Where the floor is subject to impact from moving loads, or to wear, the slab thicknesses shall be increased ½ in. If a floor or covering ½ in. or more in thickness, not included in the structural slab, is used for a wearing surface, no increase need be made.

(e) Where the slab contains conduits or pipes, the thickness shall not be less than 1 in. plus the total overall depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to reduce the strength of the construction.

(f) Shrinkage reinforcement in the slab must be provided as required in Section 712.

708: *Moment Coefficients for Freely Supported or Slightly Restrained Continuous Beams or Slabs of Approximately Equal Span; Uniform Load:*

(a) Beams and slabs of approximately equal spans freely supported or built to act integrally with beams, girders, or other slightly restraining support, or beams and slabs built into brick or masonry walls in a manner which develops only partial end restraint, and carrying uniformly distributed loads shall be designed for the following moments at critical sections:

(1) Beams and slabs of one span,
Positive moment near center,

$$M = \frac{wl^2}{8} \quad \ldots\ldots \quad (1)$$

(2) Beams and slabs continuous for two spans only,
Positive moment near center,

$$M = \frac{wl^2}{10} \quad \ldots\ldots \quad (2)$$

Negative moment over interior support,

$$M = \frac{wl^2}{8} \quad \ldots\ldots \quad (3)$$

(3) Beams and slabs continuous for more than two spans,
Positive moment near center and negative moment at support of interior spans,

$$M = \frac{wl^2}{12} \quad \ldots\ldots \quad (4)$$

Positive moment near centers of end spans and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots \quad (5)$$

(4) Negative moment at end supports for cases (1), (2), and (3) of this section,

$$M = \text{not less than } \frac{wl^2}{24} \quad \ldots\ldots \quad (6)$$

709: *Moment Coefficients for Fully Restrained Continuous Beams or Slabs of Approximately Equal Span: Uniform Load:*

(a) Beams and slabs of approximately equal spans built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformly distributed loads, shall (except as provided in Sec. 708) be designed for the following moments at critical sections:

(1) Interior spans;

Negative moment at interior supports except the first,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots \quad (7)$$

Positive moment near centers of interior spans,

$$M = \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots \quad (8)$$

(2) End spans of continuous beams or slabs, and beams or slabs of one span;

Where I/l is less than twice the sum of the values of I/h for the exterior columns above and below which are built into the beams:

Positive moment near center of span and negative moment at first interior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots \quad (9)$$

Negative moment at exterior supports,

$$M = \frac{wl^2}{12} \quad \ldots\ldots\ldots\ldots \quad (10)$$

Where I/l is equal to or greater than twice the sum of the values of I/h for the exterior column above and below which are built into the beams:

Positive moment near center of span and negative moment at first interior support,

$$M = \frac{wl^2}{10} \quad \ldots\ldots\ldots\ldots \quad (11)$$

Negative moment at exterior support,

$$M = \frac{wl^2}{16} \quad \ldots\ldots\ldots\ldots \quad (12)$$

(b) In this section, I represents the moment of inertia which, for those calculations, shall be computed on the assumption that the member is homogeneous, neglecting the reinforcement, but including that portion of the concrete section outside of the reinforcement which is ordinarily con-

sidered as fireproofing. l and h are the span length and column height, respectively, as defined in Sec. 402 and 1102.

710: *Moment Coefficients for Continuous Beams or Slabs of Unequal Span or with Non-Uniform Loads:*

(a) Continuous beams with substantially unequal spans, or with other than uniformly distributed loading, whether freely supported or restrained, shall be designed for the maximum moments resulting from the most severe probable combination of loading and restraint. Provision shall be made where necessary for negative moment near the center of short spans which are adjacent to long spans, and for the negative moment at the end supports, if restrained.

711: *Compression Steel in Flexural Members:*

(a) Where it is necessary to introduce steel in compression in girders, beams, or slabs, such steel shall be thoroughly anchored by ties or stirrups not less than ¼ in. in size which shall be spaced not more than 8 in. apart over the distance where the compression steel is required.

712: *Shrinkage and Temperature Reinforcement:*

(a) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than 18 in.:

Floor slabs where plain bars are used 0.0025
Floor slabs where deformed bars are used 0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 in..0.0018
Roof slabs where plain bars are used 0.003
Roof slabs where deformed bars are used 0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than 12 in. 0.0022

*713: *Floors Reinforced in Two Directions:*

(a) Concrete floors supported on four sides by beams, girders, or walls, and reinforced in two directions, shall be designed as follows, using moment coefficients given in Section 708, 709, and 710 as required, except as indicated under (e).

(b) If the length of the slab exceeds one and one-half times its width, the entire load shall be carried in the short direction.

(c) In case of square panels and uniformly distributed load, one-half the live- and dead-load may be assumed as being resisted by each cross band.

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* The committee feels that this section may be too conservative. However, the additional investigation necessary to determine proper design methods requires more time than has been available.

(d) In rectangular panels of length L and breadth B, the portion of the load which shall be assumed as being supported by the slab in the short direction shall be equal to $\left(\frac{L}{B}-\frac{1}{2}\right)$ times the total load. The remainder of the load shall be assumed as being supported by the slab in the long direction. The reinforcement in the long direction shall in no case be less than that specified in Sec. 712 for shrinkage and temperature reinforcement.

(e) In placing the reinforcement account may be taken of the facts that the moment is less in the portions of the band which are adjacent and parallel to the supporting beams. In the one-quarter width of band parallel and adjacent to the beams, the computed moment may be reduced 50 per cent.

(f) Beams supporting such slabs shall be assumed to take the portion of the load as determined in (b), (c), or (d) without advantage of any reduction in live-load permitted in other sections of this code. The total load for each beam shall be assumed as uniformly distributed.

714: *Maximum Spacing of Principal Slab Reinforcement:*

(a) In slabs other than ribbed floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 712 (a).

CHAPTER 8.

SHEAR AND DIAGONAL TENSION.

801: *Shearing Unit Stress:*

(a) The shearing unit stress (v) in reinforced-concrete beams shall be computed by formula (14):

$$v = \frac{8V}{7\,bd} \qquad \text{(14)}$$

When the value of the shearing unit stress computed by formula (14) exceeds the unit shearing stress (v_c) permitted on the concrete of the web (see 306-a), web reinforcement shall be provided to carry the excess.

(b) For beams of I or T section b' shall be substituted for b in formula (14).

(c) In tile and joist construction, b may be taken as a width equal to the thickness of the concrete web plus the thickness of the vertical webs of the concrete or clay tile in contact with the joist as in Sec. 707 (c).

802: *Types of Web Reinforcement:*

(a) Web reinforcement may consist of:

(1) Vertical stirrups or web reinforcing bars;

(2) Inclined stirrups or web reinforcing bars forming an angle of 30 deg. or more with the axis of the beam.

(3) Longitudinal bars bent up at an angle of 15 deg. or more with the axis of the beam.

(b) Stirrups or bent-up bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Sec. 904.

803: *Stirrups:*

(a) Area of steel required in stirrup shall be computed by formula (15).

$$A_v = \frac{V's}{14000d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

804: *Spacing of Stirrups:*

(a) Where the shearing stress is not greater than $0.06f'_c$ the distance s between two successive stirrups measured perpendicular to the direction of the stirrup shall not exceed $\frac{3}{4}d$, and where unit shearing stress exceeds $0.06f'_c$, it shall not be greater than $\frac{3}{8}d$.

805: *Bent-up Bars:*

(a) Where there is a series of parallel bent-up bars at varying distances from the support, they shall be considered as inclined stirrups and the area required determined from formula (15).

(b) Where bent-up bars in a single plane are used for web reinforcement, the required area of the bar shall be computed by formula (16).

$$A_v = \frac{V'}{16000 \sin \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

(c) In formula (16), V' shall not exceed $0.035f'_c\, bd$ nor α be less than 15 deg. Only the center three-fourths of the inclined portion of such bar or group of bars shall be considered effective in resisting shear. Between the face of the support and the area reinforced by the bent-up bar, other web reinforcement shall be provided, except that when the distance is less than $d/2$ and the beam is designed for uniform load only, such additional reinforcement need not be provided.

806: *Combined Web Reinforcement:*

(a) Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once.

807: *Shearing Stress in Flat Slabs:*

(a) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_1 - 1\frac{1}{2}$ in. from the edge of the column capital and parallel with it, shall not exceed the following values when computed by formula (14) (in which d shall be taken as $t_1 - 1\frac{1}{2}$ in.):

(1) $0.03f'_c$ when at least 50 per cent of the total negative reinforcement passes directly over the column capital;

(2) $0.025f'_c$ when 25 per cent of the total negative reinforcement passes directly over the column capital (which is the least that shall be permitted);

(3) For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(b) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_2 - 1\frac{1}{2}$ in. from the edge of the dropped panel and parallel with it shall not exceed $0.03f'_c$ when computed by formula (14) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in two column strips must be within the width of strip directly above the dropped panel.

808: *Shear and Diagonal Tension in Footings:*

(a) The shearing unit stress computed by formula (14) on a vertical section, which lies at a distance d from the face of the supported column or pier and parallel with it, shall not exceed $0.02f'_c$ for footings with straight bars, nor $0.03f'_c$ for footings in which the bars are anchored at both ends by adequate hooks or otherwise specified in Sec. 903.

(b) In footings supported on piles, the critical section for diagonal tension shall be considered the distance $d/2$ from the face of the column or pedestal and any piles whose centers are at or within this section shall be excluded in computing the shear.

CHAPTER 9.

BOND AND ANCHORAGE

901: *Computation of Bond Stress in Beams:*

(a) Where reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken as not less than that computed by formula (17).

$$u = \frac{8V}{7\Sigma_o d} \quad \dots\dots (17)$$

(b) For continuous or restrained members, the critical section for bond for the positive reinforcement shall be assumed to be at the point of inflection, that for the negative reinforcement shall be assumed to be at the face of the support, and at the point of inflection. For simple beams, or at the outer ends of freely supported end spans of continuous beams, the critical section for bond shall be assumed to be at the face of the support.

(c) Bent-up longitudinal bars which, at the critical section, are within a distance $d/3$ from horizontal reinforcement under consideration may be included with the straight bars in computing Σ_o.

902: *Ordinary Anchorage Requirements:*

(a) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame, shall have a length of anchorage beyond the face of the supporting member sufficient to develop the full maximum tension at an average bond stress not greater

than $0.04f'_c$, for plain bars, or $0.05f'_c$ for deformed bars. Within any such span, not less than one-third of the negative reinforcements shall extend along the tension side of the beam at least to or beyond the point of inflection, and any bars not so extended shall be bent down at an angle of not more than 45 deg. with the axis of the member and made continuous with the positive reinforcement or anchored in a region of compression.

(b) Of the positive reinforcement in continuous beams, not less than one-fourth the area shall extend at the same face of the beam into the support to provide an embedment of ten or more bar diameters beyond the face of the support.

(c) For non-continuous beams not less than one-half the area of positive reinforcement shall extend at the same face of the beam into the support to provide an embedment of ten or more bar diameters.

903: *Special Anchorage Requirements:*

(a) Where increased shearing or bond stresses on account of special anchorages are permitted as specified in Section 306, anchorage of all reinforcement as required in Section 902 shall be increased to conform with the requirements of (b), (c), (d), and (e) of this section.

(b) In continuous and restrained beams, anchorage beyond points of inflection of at least one-third the area of the negative reinforcement and beyond the face of the support of at least one-third the area of the positive reinforcement, shall be provided to develop one-third of the allowable working stress in tension at average bond stresses not to exceed $0.04f'_c$ for plain bars nor $0.05f'_c$ for deformed bars.

(c) In footings, all bars shall be anchored by means of hooks at the end of the bar. The total length of bar shall be the width of the footing plus 20 bar diameters The outer face of the hook shall not be less than 3 in. nor more than 4 in. from the face of the footing.

(d) In simple beams, or at the outer ends of freely supported end spans of continuous beams, at least one-half of the tensile reinforcement shall extend along the tension side of the beam to provide an anchorage beyond the face of the support for one-third of the allowable working stress in tension at an average bond stress not to exceed $0.04f'_c$ for plain bars, nor $0.05f'_c$ for deformed bars.

(e) In cases where the design of unusual members involves the use of unit shearing stresses in excess of $0.09f'_c$, values up to $0.12f'_c$ may be used, providing the requirements of this section are fully met, that the members in which these stresses are used shall be specially designated on the plans and that these members shall be constructed under the personal supervision of the designing engineer who shall notify the commissioner of buildings at least one day in advance of the placing of the concrete in such member. When required by the commissioner of buildings, the designing engineer shall submit an affidavit certifying that he has personally supervised the construction of these members and that the design and construction was in all respects as called for on the plans and in conformity with the provisions of this code.

904: *Anchorage of Web Reinforcement:*

(a) Web reinforcement shall be anchored at both ends by one of the following methods or combination thereof, but only anchorage meeting the requirements of (1), (2) or (3) shall be used for shearing unit stresses in excess of $0.08f'_c$.

(1) Providing continuity with the main longitudinal reinforcement.
(2) Bending around the longitudinal bar or steel shape;
(3) A hook which has a radius of bend not less than 4 times the diameter of the web bar;
(4) A length of embedment sufficient to develop the stress in the stirrup by bond as provided below, provided also that the other end of the stirrup is anchored as in (1).

(b) The end anchorage of a web member not bearing on the longitudinal reinforcement shall be such as to engage an amount of concrete sufficient to prevent the bar from pulling out. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fireproofing requirements permit.

(c) The stress in a stirrup or web reinforcement bar shall not exceed a value equal to the surface area of the bar embedded within the upper or lower one-half of the beam multiplied by $0.04f'_c$ for plain bars, or $0.05f'_c$ for deformed bars, except that when wire fabric is used for web reinforcement it shall have welded intersections not farther apart than 6 in., but in no case shall the stress exceed 16,000 lb. per sq. in.

CHAPTER 10.

FLAT SLABS.

(Two-Way and Four-Way Systems with Square or Rectangular Panels.)

1001: *Limitations:*

(a) The term flat slabs as used in these regulations refers to concrete slabs, having reinforcement bars extending in two or four directions, without beams or girders to carry the load to supporting members.

(b) The moment coefficients, moment distribution, and slab thicknesses specified herein are for a series of slabs of approximately uniform size arranged in three or more rows of panels in each direction, and in which the ratio of length to width of panel does not exceed 1.33.

(c) Slabs with paneled ceiling or with dropped panels shall be considered as coming under the requirements herein given, provided the dropped panel shall have a length or diameter in each direction parallel to a side of the panel of not less than 0.35 of the panel length in that direction, and provided further that the depth of the thicker portion of the slab does not exceed one and one-half times the depth of the remainder of the slab.

(d) For structures having a width of less than three rows of panels, or in which irregular panels are used, an analysis shall be made of the moments developed in both slabs and columns. When so required, computations shall be submitted to the commissioner of buildings for approval.

1002: *Panel Strips and Principal Design Sections:*

(a) For convenience of reference, a flat slab panel shall be considered as consisting of strips as follows:

A *middle strip* one-half panel in width symmetrical with respect to the panel center line and extending through the panel in the direction in which moments are being considered;

A *column strip* one-half panel in width occupying the two quarter panel areas outside of the middle strip.

When considering moments in the direction of the width of the panel, the panel is similarly divided by strips, the widths of which are each one-half the length of the panel.

(b) The critical sections for moment calculations are referred to as principal design sections and are located as follows:

Sections for Negative Moment. These shall be taken along the edges of the panel, on lines joining the column centers, and following the circumference of the column capital.

Sections for Positive Moment. These shall be taken on the center line of the panel.

1003: *Moments in Interior Panels—General Case:*

(a) The numerical sum of the positive and negative moments in the direction of either side of a rectangular panel shall be not less than that given by formula (19).

$$M_0 = 0.09Wl\left(1 - \frac{2c}{3l}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

where M_0 = sum of positive and negative bending moments at the principal design sections, in the direction in which the length is given by l. This moment is in foot-pounds when c and l are in feet and W is in pounds.

(b) The moments in the principal design sections shall be those given in the following table of moments, except that the maximum negative moment in the column strip may be greater or less than the values given in the table of moments by not more than $0.03M_0$, provided that the sum of the moments on the principal section remains equal to M_0, and provided further that the moment in each of the three other critical design sections be modified by not more than $0.01M_0$.

MOMENTS TO BE USED IN DESIGN OF FLAT SLABS

For Interior Panels Fully Continuous

General case: all values of c: M_o given by formula (19)

Strip	Flat Slabs without Dropped Panels		Flat Slabs with Dropped Panels	
	Negative	Positive	Negative	Positive
Slabs with 2-Way Reinforcement				
Column Strip..........	$-M_c = 0.46M_o$	$+M_c = 0.22M_o$	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$
Middle Strip...........	$-M_m = 0.16M_o$	$+M_m = 0.16M_o$	$-M_m = 0.15M_o$	$+M_m = 0.15M_o$
Slabs with 4-Way Reinforcement				
Column Strip.........	$-M_c = 0.50M_o$	$+M_c = 0.20M_o$	$-M_c = 0.54M_o$	$+M_c = 0.19M_o$
Middle Strip...........	$-M_m = 0.10M_o$	$+M_m = 0.20M_o$	$-M_m = 0.08M_o$	$+M_m = 0.19M_o$

(c) The width of section at the column head shall be taken as the width of the dropped panel where used or half the width of panel where no dropped panel is used.

(d) The band width in the two-way system shall be such as to provide reinforcement over the entire one-half panel width.

(e) The band width for the direct bands in the 4-way system shall be approximately 4/10 of the panel width at right angles to the direction of the band ($0.4l_1$) and for the diagonal bands approximately 0.4 of the average span length. In proportioning the reinforcement in this system, it shall be assumed that reinforcement in the direct band resists the entire positive moment for the column strip and the two diagonal bands resist the entire positive moment for the middle strip. Reinforcement for negative moment for the column strip shall include the area of reinforcement for negative moment in the diagonal bands multiplied by the cosine of the angle between the diagonal band and the axis of the direct band considered plus the full area of the reinforcement for negative moment in the direct band. The negative reinforcement for the middle strip shall be provided independently of the diagonal bands.

1004: *Moments in Interior Panels — Special Case, $c = 0.225$ times the average span length:*

(a) For the particular case where c is equal to 0.225 times the average span length (the average of the distances center to center of columns on the two sides of the panel), formula (19) reduces to formula (19a).

$$M_o = 0.065Wl \qquad \text{(19a)}$$

(b) For two-way slab, the values of M_o may be obtained from formula (19a) and the distribution taken from the table in Sec. 1003(b).

(c) For the four-way slab with dropped panel, the following table of coefficients may be used in computing the reinforcement required in each of the bands, provided that l for the direct bands shall be the center

to center distance between columns in the direction in which the band extends, and for the diagonal bands the average value of l for the two direct bands of the panel. The moments in the table are those on *sections at right angles* to the direction of the respective bands:

BAND	LOCATION	AMOUNT
Direct	Center	$+0.012Wl$
Diagonal	Center	$+0.009Wl$
Direct	At column head	$-0.020Wl$
Diagonal	At column head	$-0.011Wl$
Top band across direct band....	Between columns	$-0\ 005Wl$

1005: *Thickness of Slabs and Dropped Panels:*

(a) For slabs without dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness of the slab t_1, in inches, shall be not less than the value given by formula (20).

$$t_1 = 0.038\ (1 - 1.44\frac{c}{l})l\ \sqrt{w'} + 1\tfrac{1}{2} \quad \text{.............} (20)$$

where w' = uniformly distributed dead and live-load, lb. per sq. ft.

(b) For slabs with dropped panels, using concrete of 2,000 lb. per sq. in. ultimate strength, the total thickness in inches at points beyond the dropped panel shall be not less than

$$t_2 = 0.02l\ \sqrt{w'} + 1 \quad \text{............................} (21)$$

(c) The dropped panel shall have a thickness not greater than $1.5t_2$ nor less than $1.25t_2$. The side or diameter of the dropped panel shall not be less than 0.35 times the side of the panel in the parallel direction.

(d) In determining minimum thickness by formulas (20) and (21), the value of l shall be the panel length center to center of the columns, on the long side of the panel. For concrete of 2,000 lb. per sq. in. ultimate strength, the slab thickness t_1 or t_2 shall in no case be less than $l/32$ for floor slabs, and not less than $l/40$ for roof slabs.

(e) Where concretes of higher ultimate strengths than 2,000 lb. per sq. in. are used, the thickness given by formulas (20) and (21) and the limiting thicknesses may be reduced by multiplying by the factor $\sqrt[3]{\frac{2{,}000}{f'_c}}$, in which f'_c is the ultimate strength of the concrete to be used.

1006: *Limiting Percentages of Reinforcement:*

(a) The ratio of reinforcement for negative moment in the column strip shall not exceed the values of p calculated for balanced reinforcement, that is, the amount of reinforcement for which both the steel and the concrete are stressed to the full amount permitted by Sec. 306 and 307. Any reinforcement in excess of this amount shall not be included in the calculation. In computing the ratio of reinforcement for negative moment in the column strip, the width of section shall be taken as in Sec. 1003 (c).

In the case of four-way design, the steel area shall consist of the area of steel for negative moment as defined in 1003(e).

(b) The ratio of flat slab reinforcement in any strip shall not be less than 0.0025. Bars shall not be spaced farther apart than 1½ times the slab thickness.

1007: *Point of Inflection:*

(a) In the middle strip the point of inflection for slabs without dropped panels shall be assumed at a line $0.33l$ distant from the center of the span and for slabs with dropped panels $0.3l$ distant from the center of the span.

(b) In the column strip, the point of inflection for slabs without dropped panels shall be at a line $0.33\ (l - c)$ distant from the center of the panel and $0.3\ (l - c)$ for slabs with dropped panels.

1008: *Arrangement of Reinforcement at Column Heads—Two- and Four-Way Systems:*

(a) In both two- and four-way systems, provision shall be made for securing the reinforcement in place so as to resist properly not only the critical moments, but also the moments at intermediate sections. The full area of steel required for negative moment at the column head shall be continued in the same plane close to the upper surface of the slab to the edge of the dropped panel, but in no case less than a distance $0.2l$ from the center line of column. Lapped splices shall not be permitted at or near regions of maximum stress except as described in Sec. 505.

1009: *Arrangement of Reinforcement—Two-Way System:*

(a) For column strips at least four-tenths of the area of steel required at the section for positive moment in the column strip shall be of such length and so placed as to reinforce the negative moment section at the two adjacent column heads. These bars, and any other bars for negative reinforcement shall extend into the adjacent panel to a point at least $0.05l$ beyond the point of inflection. Not less than one-third of the bars used for positive reinforcement in the column strip shall extend into the dropped panel at least twenty diameters of the bar, but not less than 12 in. or in case no dropped panel is used, shall extend to within $0.125l$ of the center line of the columns or the supports. The balance of the bars for positive reinforcement in the column strip shall extend at least $0.33l$ on either side of the center line of panel.

(b) For the middle strip at least one-half of the bars for positive moment shall be bent up and extend over the main bands at both sides of the panel to a point at least $0.25l$ beyond the center line of columns. The location of the bends shall be such that for a distance $0.15l$ for slabs with dropped panels, (or $0.125l$ for slabs without dropped panels), on each side of the center line of columns, the full reinforcement required for negative moment will be provided in the top face of the slab. The full reinforcement for positive moment in the middle strip shall extend in the bottom face of the slab to a point at least $0.3l$ on either side of the panel

center line, and at least 50 per cent of it shall extend to points $0.325l$ on either side of the panel center line for slabs with dropped panels, or $0.35l$ for slabs without dropped panels.

1010: *Arrangement of Reinforcement—Four-Way System:*

(a) For direct bands, all provisions governing the placing of steel in column strips in two-way systems apply as well to the direct bands in four-way systems.

(b) For diagonal bands, at least four-tenths of the area of steel required at the section for positive moment shall be of such length and so placed as to reinforce the negative moment section at the two diagonally opposite column heads. These bars and any other bars for negative reinforcement shall extend into the adjoining panel to points at least $0.4l$ beyond a line drawn through the column center perpendicular to the direction of the band. The straight bars for positive moment in the diagonal bands shall not be shorter than the longer straight bars in the direct bands.

(c) For negative moment in the middle strip, the required steel shall extend not less than $0.25l$ on either side of the column center line.

1011: *Wall and Other Irregular Panels:*

(a) In wall panels and other panels in which the slab is non-continuous on one edge, the maximum positive moments on the principal design sections parallel to the discontinuous edge (reinforcement perpendicular to that edge) shall be increased by 25 per cent.

(b) The positive moment reinforcement perpendicular to the discontinuous edge shall extend to this edge and have an embedment of at least 6 in. in spandrel beams or columns. All negative moment reinforcement shall be bent or hooked at spandrel beams or columns to provide adequate bond resistance.

(c) At the wall or discontinuous edge the negative moment in the column strip shall be taken as not less than 90 per cent and in the middle strip not less than 62½ per cent of the corresponding moments for a normal interior panel as given in the table of Sec. 1003(b).

(d) Where there is a beam or a bearing wall at the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30 per cent greater than the negative moment specified in Sec. 1003(b) for a middle strip. The half column strip adjacent and parallel to and lying on either side of the beam or wall shall be designed to resist moments at least one-fourth of those specified in Sec. 1003(b) for a column strip. The beam or wall in such cases shall be designed to carry a uniformly distributed load equal to one-fourth of the panel load on either side in addition to the loads directly imposed upon it.

1012: *Panels With Marginal Beams:*

(a) In panels having marginal beam on one edge or on each of the two adjacent edges, the beam shall be designed to carry at least the load

superimposed directly upon it, exclusive of the panel load. A marginal beam which has a depth greater than 1½ times the minimum slab thickness, shall be designed to carry, in addition to the load superimposed directly upon it, a uniformly distributed load equal to at least ¼ of the total live and dead load for which the adjacent panel or panels are designed. Slabs supported by marginal beams on opposite edges shall be designed as freely supported slabs for the entire load.

(b) The half column strip adjacent to and parallel with marginal beams, having a depth not greater than 1½ times the minimum slab thickness, shall be designed to resist half the moment specified for a full column strip.

(c) In wall panels having exterior columns where brackets, (the faces of which make an angle with the face of the column, projected upward, of not more than 45 deg.) are used in place of capitals, the value of c in the direction in which the bracket extends may be taken as twice the distance from the center of the column to a point where the structural portion of the bracket is 1½ in. thick, and averaged with the value of c for an interior column capital in the computations for moment in formula (19). The value of c for column strips parallel and adjacent to a non-continuous edge of a slab where either no marginal beam is used, or where the beam used is not deeper than 1½ times the minimum slab thickness, should be taken as equal to the width of the wall column if no bracket is used in this direction.

(d) The value of c for column strips parallel and adjacent to marginal beams having a depth greater than the thickness of the slab at the wall columns, shall, if no bracket is used in this direction, be taken as equal to the width of the wall column plus twice the difference between the depth of the beam and the depth of the slab through the dropped panel. This value of c is to be used in calculating the $-M_c$ and $+M_c$ for the half column strip parallel and adjacent to the marginal beams only. This half column strip should be designed to resist a moment at least one-fourth as great as that specified for a column strip in the Table of Moments.

(e) It shall be permissible to omit the dropped panels at wall columns provided the design complies with the requirements of Section 1003(b) and 1006(a) for slabs without dropped panels.

1013: *Openings in Flat Slabs:*

(a) Openings of any size may be cut through the floor in the area common to two intersecting middle strips, provided the total positive and negative resisting moments be maintained as required in Sec. 1003(b) and that these total positive and total negative moments be redistributed between the remaining principal design sections to meet the new conditions.

(b) In any area common to two column strips, not more than one opening shall be allowed and the greatest dimension of such an opening shall not exceed $1/20l$.

(c) In any area common to one column strip and one middle strip, openings shall not interrupt more than one-quarter of the bars in either

strip and the equivalent of the bars so interrupted shall be provided by extra steel on both sides of the opening.

(d) Any opening larger than described above shall be completely framed on all sides with beams to carry the loads to the columns.

CHAPTER 11.

REINFORCED-CONCRETE COLUMNS AND WALLS.

1101: *Limiting Dimensions:*

(a) Unless designed as long columns under the provisions of Sec. 1108, reinforced-concrete columns shall not be longer than eleven times the least lateral dimension. Principal columns in buildings shall have a minimum diameter or thickness of 12 in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of 6 in.

1102: *Unsupported Length of Columns:*

(a) The unsupported length of reinforced-concrete columns shall be taken as:

(1) In flat-slab construction the clear distance between the floor and under side of the capital;

(2) In beam-and-slab construction, the clear distance between the floor and the under side of the shallowest beam framing into the column at the next higher floor level;

(3) In floor construction with beams in one direction only, the clear distance between floor slabs;

(4) In columns supported laterally by struts or beams only, the clear distance between consecutive pairs (or groups) of struts or beams, provided that to be considered an adequate support, two such struts or beams shall meet the column at approximately the same level and the angle between the two planes formed by the axis of the column and the axis of each strut respectively is not less than 75 deg., nor more than 105 deg.

(b) When reinforced-concrete brackets are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by the depth of the bracket, provided the width of the bracket is at least equal to that of the beam and not less than one-half of the column.

1103: *Design of Spiral Columns:*

(a) The permissible axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core, shall not be greater than that determined by formula (22).

$$P = A_c\,[1 + (n - 1)\,p]\,f_c \qquad (22)$$

in which A_c is the area within the outer circumference of the spiral hooping, and the values of f_c are as given in Sec. 306, or as may be found for intermediate values of p by interpolation, or in general, by the formula,

$$f_c = [300 + (0.10 + 4p)f'_c\ ] \qquad (22a)$$

(b) The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and of an effective cross sectional area not less than 0.01, nor more than 0.06 of that of the core. The number of longitudinal bars concentrated in the ring at the periphery of the core shall be governed by the spacing requirements of Section 504(a). If all the bars cannot be placed at the periphery of the core, the bars within shall be stayed at intervals of 24 in., and shall not be nearer to the outer ring than two-tenths times the core diameter. When the ratio of reinforcement in a spirally reinforced column is greater than 0.04, special placing drawings illustrating the proper distribution of steel shall be submitted with the detail plans. Splices in longitudinal reinforcement shall provide a lap of at least 24-bar diameters for deformed bars, and 30 diameters for plain bars.

(c) The ratio of the spiral reinforcement shall be not less than one-fourth the ratio of the longitudinal reinforcement. Spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. At the ends of all spirals and at points of splice, the outside diameter shall be maintained. The spacing of the spirals shall not be greater than one-sixth of the diameter of the core and in no case more than 3 in.

(d) Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core which shall have a minimum thickness of 1½ in.

1104: *Design of Columns with Lateral Ties:*

(a) The permissible axial load on columns reinforced with longitudinal bars and separate lateral ties shall not be greater than that determined by formula (23):

$$P = 0.225 f'_c A_g [1 + (n - 1)p] \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

(b) The ratio of longitudinal reinforcement shall not be less than 0.005 nor shall the ratio considered in the calculations be more than 0.02 of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of ⅝ in., placed with clear distance from the face of the column not less than 2 in., nor more than 3 in. Splices in longitudinal reinforcement shall provide a lap of at least 24-bar diameters for deformed bars, and 30 diameters for plain bars.

(c) Lateral ties shall be at least ¼ in. in diameter spaced not more than 12 in. apart. In columns of rectangular section, cross ties shall be arranged to afford support to the vertical bars at intervals not greater than the shorter side of the section, but such interval need not be less than 12 in. in any case.

1105: *Bending in Columns:*

(a) The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint, and shall be provided for in the design.

(b) In flat-slab construction, the least dimension of the column shall be not less than one-fifteenth of the average center to center span, nor less

than 16 in. For known eccentric loads or unequal spacing of columns, computations of moments shall be made accordingly. Wall columns in flat-slab construction shall be designed to resist a bending moment of $Wl/35$. Any counter moment due to the weight of the structure that projects beyond the column center line may be deducted from the moment computed as just described. Resistance to the bending moments shall be divided between the columns immediately above and below in direct proportion to the values of their ratios of I/h (see Sec. 709 and 1102).

(c) The recognized methods shall be followed in calculating the stresses due to combined axial load and bending. The column section shall not be less than that required where axial load alone is considered. The limiting combined unit stresses shall be as follows:

(1) Columns with spiral reinforcement,
$[300 + (0.10 + 4p)f'_c] + 0.15f'_c$
(2) Columns with lateral ties $0.3f'_c$. The total amount of reinforcement considered in the computations shall not be more than 4 per cent of the total area of the column.
(3) Tension in longitudinal reinforcement due to bending on the column shall not exceed 16,000 lb. per sq. in.

(d) Where the allowable unit stress in columns is increased (to provide for combined axial load and bending) and wind loads are also added, the total shall still come within the allowable values specified for wind loads in Sec. 604.

1106: *Composite Columns:*

(a) The permissible load on composite columns in which a structural steel or cast-iron column is thoroughly encased in a concrete core reinforced with not less than 0.02 nor more than 0.04 longitudinal reinforcement in the form of bars arranged at the periphery of the core, nor less than 0.01 of spiral reinforcement meeting the requirements for spirals of Sec. 1103 (c), shall be based on a certain unit stress for the steel or cast-iron section plus a unit stress of $0.25f'_c$ on the net area of the concrete within the outer circumference of the spiral hooping enclosing the core. The longitudinal and spiral reinforcement ratios stated shall be based on the total core area enclosed within the outer circumference of the spiral hooping.

(b) The unit compressive stress on the steel section shall not exceed 15,000 lb. per sq. in. Where the steel section is required to carry construction or other loads prior to its encasement in concrete, the stress shall not exceed that given by formula (24).

$$f_r = \cfrac{18{,}000}{1 + \cfrac{h^2}{18{,}000\,R^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (24).$$

(c) The unit stress on the cast-iron section shall not exceed 9,000 lb. per sq. in. Where the cast-iron section is required to carry construction, or other loads prior to its encasement in concrete, the stress shall not exceed that given by formula (25).

$$f_r = 12{,}000 - 60\frac{h}{R} \qquad (25).$$

(d) The unit stress on the longitudinal reinforcement shall be $0.25nf'_c$.

(e) The diameter of the cast-iron section shall not exceed one-half the diameter of the core, nor shall its total area exceed 12 per cent of the core area, (area included within outer circumference of the spiral hooping). The dimension of the structural steel section shall be such as to provide at least 3 in. between the spiral and the corners of the section and its area shall not exceed 12 per cent of the core area. Metal columns shall be accurately milled at splices and positive provision shall be made for alignment of one column above another. The spiral reinforcement shall be not less than 0.01 of the volume of the core, and shall conform in quality, spacing, and other requirements to the provisions for spirals in Sec. 1103 (c).

(f) In composite columns, provision shall be made at the base to transfer the load from the middle section at safe unit stresses in accordance with Section 1205. The base of the metal section shall be designed to transfer the load from the entire composite column to the foundation, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of the load from the reinforced-concrete section of the column by means of bond on the vertical reinforcement, and by direct compression from the concrete. At the top of the metal section, provision shall be made to receive the full load to be transferred to the metal section at this point. At points in the structure below this, where the load on the metal section is increased, positive means, consisting of cast or built-up brackets rigidly attached to the metal section, shall be provided to receive the increase in load.

(g) Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and the metal column shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35f'_c$, unless special brackets are arranged on the metal column to receive directly the beam or slab load.

1107: *Combination Columns:*

(a) Structural steel columns of any rolled or built-up section wrapped with the equivalent of No. 8 U. S. standard gage wire spaced 4 in. on center and encased in concrete not less than 2 in. thick over all of metal except rivet heads and connections will be permitted to carry a load equal to $(1+A_c/100A_s)$ times permissible load for unencased steel columns.

(b) The permissible load for unencased steel columns shall be determined by formula (24), provided the structural steel column acting independently of the concrete shall have sufficient capacity to carry all dead loads which will be placed thereon, and provided the quality of the concrete is such that it shall show a compressive strength of at least 2,000 lb. per sq. in. at 28 days when tested in accordance with Sec. 201 (c).

1108: *Long Columns:*

(a) The permissible working load on the core in axially loaded spiral or composite columns which have a length greater than 50 times the least radius of gyration of the column core ($50R$) shall not be greater than that determined by formula (26).

$$\frac{P'}{P} = 1.50 - \frac{h}{100R} \qquad \text{(26)}$$

(b) The permissible working load on axially-loaded tied columns, which have a length greater than 40 times the least radius of gyration of the column section ($40R$), shall not be greater than that determined by formula (26a).

$$\frac{P'}{P} = 1.33 - \frac{h}{120R} \qquad \text{(26a)}$$

(c) The radius of gyration of a column shall be computed from the concrete area used in design and the transformed section of the longitudinal steel area; that is, the actual area of steel multiplied by n.

1109: *Monolithic Walls:*

(a) Reinforced-concrete *bearing* walls shall have a thickness of at least one twenty-fifth (1/25) of the unsupported height or width, provided, however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of greater thicknesses. Working compressive stresses in such walls shall not exceed $0.0625f'_c$ when the wall is 25 times the thickness in height, increasing proportionately to $0.125f'_c$ when the wall is 15 times the thickness or less in height.

(b) The lateral support for such walls shall consist of a fire-resistive floor when the framing is on one side of the wall only, but may be of a fire-resistive or of a non-fire-resistive type where framing is on both sides of the wall, provided that for residences, wood-frame construction properly tied may be used as support.

(c) In fire-resistive buildings, reinforced-concrete *bearing* walls shall have a thickness at least equal to the values shown in the table of minimum wall thicknesses given at the end of this section, except that exterior basement walls shall not be less than 8 in. thick.

(d) In fire-resistive buildings, bearing walls shall be reinforced with an area of steel in each direction, vertical and horizontal, at least equal to 0.0025 times the cross-sectional area. Walls 8 in. or more in thickness shall have half the steel at each face of the wall. The bars shall not be farther apart in either direction than 18 in., regardless of whether the

steel is disposed in one or two layers, nor shall less than the equivalent of ⅜-in. round bars be so used. The vertical steel shall not be relied on to carry load unless tied and arranged as in reinforced-concrete columns.

(e) All bearing walls shall be designed for any lateral pressure to which they are subjected. Eccentric loads and wind stresses shall be fully provided for. In such designs, the stresses for flexure as given in Sec. 306 shall govern.

(f) In non-fire-resistive buildings, exterior bearing walls may be of reinforced concrete, subject to the provisions of this section, when increased 50 per cent in thickness over those referred to in (c). In such walls, the amount of reinforcement in each direction, horizontal and vertical, shall be at least 0.002 times the cross-sectional area. The steel shall be distributed half to each face of the wall with a maximum bar spacing of 24 in.

(g) In buildings of skeleton construction, panel or other walls supported on the structural frame shall not be less than 5 in. thick, nor less than one-thirtieth (1/30) of the horizontal distance between columns, cross walls, or equivalent anchorage. Such walls shall be reinforced in the same manner as bearing walls in fireproof buildings, (see (d) above).

(h) Stairway and elevator enclosures in all classes of buildings may be built of reinforced concrete, when the wall thicknesses are in accordance with the requirements of (c) and (g) and reinforcement in accordance with (d).

MINIMUM WALL THICKNESS, IN INCHES, IN STORY INDICATED.

No. of Stories	Basement	1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
1	6	6	..	..	..	..	..	..	..	..	..
2	7	6	6	..	..	..	..	..	..	..	..
3	8	7	7	6	..	..	..	..	..	..	..
4	8	8	7	7	6	..	..	..	..	..	..
5	9	8	8	7	7	6	..	..	..	..	..
6	9	9	8	8	7	7	6	..	..	..	..
7	10	9	9	8	8	7	7	6	..	..	..
8	10	10	9	9	8	8	7	7	6	..	..
9	12	10	10	9	9	8	8	7	7	6	.
10	12	12	10	10	9	9	8	8	7	7	6

CHAPTER 12.

FOOTINGS.

1201: *Loads:*

(a) Footings resting directly on soil or on piles shall be proportioned as to area or number of piles on the basis of the total column load plus the weight of the footing itself. For computations of moments and shears, an upward reaction per unit area or per pile shall be based on the total column load (not including the weight of the footing itself) divided by the area or by the number of piles.

1202: *Sloped or Stepped Footings:*

(a) Footings in which the thickness has been determined by the requirements for shear as specified in Sec. 808, may be sloped or stepped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

1203: *Bending in Footings:*

(a) The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast-iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

(b) The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths of the projection of the footing from the face of the column. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by formula (27).

$$M = \frac{w}{2}(a + 1.2c)c^2 \quad \text{.........................} (27)$$

(c) For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column.

1204: *Shearing and Bond Stresses:* See Sec. 808, also Sec. 901 to 904.

1205: *Transfer of Stress at Base of Column:*

(a) The compressive stress in longitudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by dowels. There shall be at least one dowel for each column bar, and the total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the column and into the pedestal or footing not less than 30 diameters of the dowel bars for plain bars, or 24 diameters for deformed bars.

(b) The permissible compressive unit stress on top of the pedestal or footing directly under the column shall be not greater than that determined by formula (28).

$$r_a = p_a \sqrt[3]{\frac{A}{A'}} \quad \text{.................................} (28)$$

(c) The value of p_a shall not exceed 0.25 f'_c for plain concrete. When lateral reinforcement in the form of spiral or hoops is provided, the value of p_a for the area within the spiral may be increased $(1 + 2.5np')$ times that for plain concrete, but no area outside the outer face of the spiral shall be considered. Where piers are designed as columns, the value of p_a shall be computed by the proper column design formula.

(d) In no case shall the total load computed by formula (28) be taken as greater than the load computed, using a stress equal to p_a, on the gross area of the pedestal, pier, or footing at a point below special reinforcing provided at the top.

(e) Where the loaded area is not central on the top of the pedestal pier, or footing, the total area A shall not be taken as greater than the area of the largest circle that can be drawn about the load as a center and lying entirely within the top of the pedestal, pier, or footing.

(f) Where lateral reinforcement is provided to increase the value of p_a, it shall extend to within 3 in. of the top of the pedestal, pier, or footing and to a depth equal to the diameter of the spiral, and the loaded area shall lie at the center of the spiral or hoops. The pitch of the spiral or the spacing of the hoops *in the clear* shall not be less than 2 in., nor more than 5 in. The designed pitch shall be maintained by at least four spacers securely fastened to each spiral turn or hoop. The ratio of lateral reinforcement shall not exceed 0.015.

(g) In sloped or stepped footings, A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

1206. *Pedestals without Reinforcement:*

(a) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25f'_c$, unless reinforcement is provided and the member designed as a reinforced-concrete column.

(b) The depth of a pedestal or pedestal footing shall not be greater than three times its least width and the projection on any side from the face of the supported member shall not be greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars having a cross-sectional area of not less than 0.20 sq. in. per ft. in each direction, placed 3 in. above the top of the piles.

Discussion—Standard Building Regulations.*

F. R. McMillan.—Our committee has been in session almost continuously since Monday morning and we have covered a good many things and I have not had the opportunity to bring before you, by way of introduction, some things I had hoped to say at this time. My comments, therefore, must deal largely with the subject-matter of the code as we have it before us, and with some amendments which the committee wishes to present at this time. Mr. McMillan.

The occasion for these amendments is simply this: A year ago we presented a tentative draft of a proposed building code which was modified only slightly from that presented in 1925. We were encouraged at that time by some preliminary conversation with the committee of the Concrete Reinforcing Steel Institute to believe that we might, in co-operation with it, arrive at a common code. It had before the public a code which had been approved by its members.

On most of the essentials, this code and ours were in agreement. However, on the method of approach or treatment they were not at all similar, and it seemed a worthy enterprise to see if the two committees could not formulate a code which met the essential requirements on all matters and as many of the non-essentials as could be agreed upon.

As all of you know, many of the provisions that enter into documents of this kind are more or less the result of individual viewpoints. A dozen men try to prepare a building code or a specification. Each one sees every section, every item, from some outstanding experience in his career. He has some particular provision that he thinks should be used, and the final draft must be a compromise on all of these non-essentials. It seemed to the members of both committees that the advantage of having one code upon which organizations of this kind could agree was worth a great deal in presenting a united front to building code committees faced with the task of revising the code in a given city.

The code we are submitting tonight is our effort to achieve this end. It has meant a good deal of hard work on the part of both committees,

* (Editor's Note.—The stenographer's transcript of the discussion of and action taken on the report of Committee E-1 fills 81 typewritten sheets. This consists very largely in a record of Mr. McMillan's motions to amend the proposed code, the seconding of the motions from the floor, the call for discussion, putting the motion and the statement that each amendment was adopted without opposition. As there were forty-odd amendments, in most instances adopted as proposed by the committee action between preprint and convention, the Editorial Committee regards it a waste of printed space to report this session in full. The preprinted report has, therefore, been revised to agree with all amendments adopted in convention and as so revised is preprinted in the foregoing pages. The record which follows includes such discussion as indicates opinions and viewpoints in opposition to those of the committee and information which might have a bearing in the future consideration of the tentative code as adopted.)

but we do not submit it as a perfect code. We are going to leave to some future generation the novelty of writing the first perfect building code. We recognize its limitations and its errors. It is so radically different in form, though not in essence, from the code which we presented last year to this Institute, that we cannot offer it as a standard. It is offered as a revised tentative standard to be substituted for the one which has been before the committee during the past year.

As you study through the two drafts you will find that a good many changes have been made, but you will also find, except in one or two matters to which I will immediately call your attention, that on all essentials the two documents are the same. There are four points to which I will call your attention which are printed on page 2 of the introduction to this code. It is worth pausing to see what they are. (1) The discrimination in the 1927 report against the use of rail-steel reinforcement has been removed. That discrimination, you will recall, had to do with limiting the use of rail steel to straight bars, or in the case of bent bars, to those of sizes less than ¾ in.. (2) The unit tensile stresses in intermediate grade billet-steel reinforcing bars and in rail-steel reinforcing bars have been increased from 18,000 to 20,000 lb. per sq. in., while the stress in all web reinforcement has been decreased to 16,000 lb. per sq. in. (3) The method of specifying concrete has been changed in detail, though in its essence it remains the same, except that provision is made for the use of concrete with strength in excess of 3000 lb. per sq. in. where very rigid control is assured. (4) The maximum permissible shearing stress on concrete beams has been reduced from 0.12 f'_c to 0.09 f'_c with the provision that beams or girders may be used with shearing stresses greater than 0.09 f'_c (but not greater than 0.12 f'_c) when such beams or girders are clearly indicated on the plans, and when the designer shall personally supervise the construction of such members.

The amendments which we are proposing are made necessary by the size of the task which we have attempted. It has been no small task. We began early in the summer. We had many sessions, several of the full committee, many of small groups from each of the two committees. We have submitted this complete report to the full committee for letter ballot and comments. The replies have made necessary certain additions. These together with other little points that have come up in our meetings this week make it necessary to submit some amendments at this time. We regret that it was not done in time to have a clear copy, with all of our amendments incorporated, in your hands prior to the convention.

Before I present the amendments, one at a time, I am going to ask Mr. Zabriskie, president of the Gabrielle Steel Co. and chairman of the Committee of the Concrete Reinforcing Steel Institute, to say a word. His committee has had an important part in this document.

Mr. Zabriskie. W. F. ZABRISKIE.—I can add very little to what Mr. McMillan has already told you. The Concrete Reinforced Steel Institute is, as you know, a commercial organization. It is like other trade associations, organized

to promote the interests of its members. It was, however, recognized, soon after the Institute was founded, that it would be very desirable, if possible, to secure some uniformity among the various regulations. To that end the Committee on Engineering Practice formulated a code and published it. It was only shortly after that, I believe, that this tentative standard issued by Committee E-1 appeared. It appeared absurd to have two documents, both of which claimed to be authoritative and both of which were urged for adoption on various municipalities.

Speaking for our Institute, I can say that we have seen from the beginning the desirability of trying to agree on a common standard with the American Concrete Institute, and we have had the utmost co-operation from Mr. McMillan and the most courteous treatment from both the chairman and the members of your Committee E-1. Wherever our ideas differed from yours, we always received a courteous hearing in the consideration of the ideas, and whether the ideas were finally adopted or not, we know that they were given due consideration. We believe that the document as now presented is a great improvement on our code, and on both codes. I believe that if it is adopted, the reinforced-concrete industry, speaking broadly, will have accomplished a tremendous achievement, and one I would like to say I believe rests principally with your chairman, Mr. McMillan.

F. R. McMILLAN.—After these amendments, the committee is proposing, have been disposed of, the report will be under discussion, and, of course, it will be in order for any amendment to be submitted from the floor. It will, I think, expedite matters if we can have these amendments disposed of rapidly. Many of them will be merely minor editorial corrections over which there should be little or no discussion, although we will appreciate any discussion on any of the matters, and it is not our purpose to rush anything through. **Mr. McMillan.**

We have a few changes in definitions. It is the hardest thing in the world to get twenty-five men to agree on a definition of anything, and we have been obliged to make some changes because new slants have been brought up and the modified definitions are an improvement and in many cases a vital necessity to a proper interpretation of the code.

F. R. McMILLAN.—The first major amendment which we have to offer is under the first chapter, and the amendment is to add section 103, paragraph (a), special systems of reinforced concrete. "The sponsors of any system of reinforced concrete which has been in successful use and the design of which is either in conflict with these provisions or not covered by them," etc. **Mr. McMillan.**

JOSEPH H. CHUBB.—You specify successful use. You could not introduce any new system if you had that provision in it. **Mr. Chubb.**

MR. MITCHELL.—I might suggest successful use or test, because if it is tested out I see no reason why it should be required to be in successful use. I offer that as an amendment. (Seconded and adopted.) **Mr. Mitchell.**

Mr. McMillan. F. R. McMILLAN.—I will take a little liberty with the reading of the amendment. The amended motion will now read "The sponsors of any system of reinforced concrete which has been in successful use or the adequacy of which has been shown by tests and the design of which is either in conflict with these provisions," etc. (The amendment was unanimously adopted.)

Mr. Rae. JAMES C. RAE.—I would like to get back a step to where we mentioned structural steel, composite or combination columns. The Committee carefully defined a composite column, but very much ignored a combination column. Is not a combination column worthy of a definition?

Mr. McMillan. F. R. McMILLAN.—There is no definition of it. Article 1107 covers combination columns. Perhaps a motion to request the committee to supply a definition of combination columns would cover your point.

Mr. Rae. JAMES C. RAE.—I make that motion.

Mr. Mitchell. MR. MITCHELL.—May I suggest, too, that in the specification for stresses for composite columns, the word combination be omitted, because it provides only for composite columns, and not for combination columns? A separate line should be added for combination columns.

Mr. Lindau. CHAIRMAN LINDAU.—We want now a definition for a combination column. (The motion was seconded and unanimously adopted.)

Mr. McMillan. F. R. McMILLAN.—The next amendment is on section 506. In paragraph (b), the first sentence is amended. There is no change in intent, it is merely in the manner of expression; "In fire-resistive construction, metal reinforcement shall be protected by not less than 1 in. of concrete in slabs and walls and not less than 1½ in. in beams, girders and columns, provided coarse aggregate which is free from disruptive action under high temperature, as for example limestone or traprock, is used. When impracticable to obtain aggregate of this grade, the protective covering shall be ½ in. thicker and shall be reinforced with metal mesh having openings not exceeding 3 in., placed 1 in. from the finished surface," etc.

Mr. Hatt. W. K. HATT.—I wonder if that provision just read is definite enough to incorporate in a building code, and to be interpreted by the commissioner of buildings? I think this paragraph should be referred back to the committee for consideration.

Mr. Ingberg. S. H. INGBERG.—I believe that course would possibly be the best in this case. I do not believe the material you have in here really covers the ground. I suggest that this paragraph be given further consideration.

(The amendment as proposed by the Committee was then adopted.)

Mr. Lindau. CHAIRMAN LINDAU.—Dr. Hatt, do you make a separate motion that this matter be referred back to the committee for further consideration?

Mr. Hatt. W. K. HATT.—Yes sir.

The motion of Professor Hatt was unanimously adopted.

CHAIRMAN LINDAU.—Is there any discussion now, gentlemen? You can talk on the whole report, if you like. Mr. Lindau.

C. E. LOCKE.—I would like to bring up the specification 1109 (a) monolithic walls. It refers to a thickness of at least one twenty-fifth of the unsupported height. I was going to suggest that we say height or width. Mr. Locke.

F. R. MCMILLAN.—A thickness of at least one twenty-fifth of the unsupported height or width? Mr. McMillan.

C. E. LOCKE.—Yes. (Motion seconded and carried.) Mr. Locke.

F. E. RICHART.—On pages 35, 36, 37 and 38 the expression "safe load" on columns appears. In a building code it would be better to say permissible load. Mr. Richart.

The motion was adopted.

E. E. HUGHES.—I move the formal adoption of the report of the committee as amended and read by the chairman at this time, as a tentative report. Mr. Hughes.

The motion was seconded by Mr. Rae and unanimously adopted.

Standard Building Units.

Report of Committee P-1.

Activities of your committee on Standard Concrete Building Units during 1927 show favorable progress on six major objectives. A series of preliminary tests was made for investigating the effects of various methods of tamping and feeding employed in the manufacture of dry tamped concrete masonry units. Acknowledgment is made to Messrs. Martin and George Hammerschmidt of the Elmhurst-Chicago Stone Company for furnishing the necessary materials and for the use of their plant in which the specimens were made.

The program of tests includes two gradings of aggregates, two consistencies, four methods of feeding concrete into the mold and four variations in numbers of tamps, aggregating twelve variables combined to give 56 different conditions. The discussion of tests is a part of this report and appears as Appendix I.

The committee has voted to continue the specifications for concrete brick as tentative standard for another year. We hope that some means of co-operation can be established with the American Society for Testing Materials Committee on concrete brick. Their specifications differ from ours, but the differences do not seem to be irreconcilable. Several of our committee members have submitted pertinent information on this subject. It has been proposed to appoint a sub-committee on concrete brick.

The committee has voted to omit the heavy load-bearing classification from the Standard Specifications for Concrete Block and Building Tile and also revise the requirement for non-load-bearing block and tile as follows:

Tentative Amendment of Standard Specifications for Concrete Block and Concrete Building Tile.

P-1A-26T.

PRESENT SPECIFICATIONS.

I. General.

3. According to the strength in compression 28 days after being manufactured or when shipped, concrete block and concrete tile shall be classified as heavy load bearing, load bearing, and non-load bearing on the basis of the following requirements:

Name of Classification	COMPRESSIVE STRENGTH, LB. PER SQ. IN. OF GROSS CROSS-SECTIONAL AREA AS LAID IN THE WALL — Average of 3 or more units	COMPRESSIVE STRENGTH, LB. PER SQ. IN. OF GROSS CROSS-SECTIONAL AREA AS LAID IN THE WALL — Minimum for individual units
Heavy load-bearing block or tile	1,200	1,000
Medium load-bearing block or tile	700	600
Non-load-bearing block or tile	250	200

REVISED SPECIFICATIONS.

3. The average compressive strength of 3 or more concrete block or concrete building tile in lb. per sq. in. of gross cross-sectional area as laid in the wall shall not be less than 700 lb., no one unit falling below 600 lb. 28 days after being manufactured or when shipped.

Where ever concrete block or building tile are used to carry unusual heavy loads, the average compressive strength of 3 or more of these units 28 days after being manufactured or when shipped in lb. per sq. in. of gross cross-sectional area as laid in the wall shall be at least 10 times the figured superimposed load to be applied.

Non-load-bearing concrete block and concrete tile shall have sufficient strength necessary to prevent excessive breakage during delivery and handling.

As the standing committee on standard sizes of concrete masonry units of the Division of Simplified Practice, U. S. Department of Commerce, we have tabulated the results of a national survey of the concrete products industry conducted during 1927 by the Portland Cement Association to show the number of different sizes of block and tile now being produced. The survey includes reports from 4,198 manufacturers which show that there are 59 different sizes of concrete building units now on the market. This is a marked reduction over the number of sizes in use when work on standardization began. Although the per cent production of each size has not been accurately recorded, it can be said that progress toward standardization of sizes for concrete masonry units is decidedly favorable. Standard sizes of concrete block and building tile are given below:

Block Width, Height, Length	Tile
6 x 7¾ x 15¾	5 x 3¾ x 12
8 x 7¾ x 15¾	5 x 8 x 12
10 x 7¾ x 15¾	5 x 12 x 12
12 x 7¾ x 15¾	3 x 12 x 12
	4 x 12 x 12
	6 x 12 x 12
Tolerance:	8 x 12 x 12
—¼—⅛—⅛	10 x 12 x 12
	12 x 12 x 12
	3 per cent permissible variation

We would recommend to the Division of Simplified Practice, U. S. Department of Commerce, that no changes be made in their standard sizes for these units during the ensuing year.

Your committee recommends that Tentative Specifications for Concrete Manhole and Catch Basin Block (P-1C-27T) be continued as tentative.*

J. F. WINKLER,
Secretary.

* The convention adopted an amended title for the specification: "Tentative Specifications for Concrete Sewer Manhole and Catch Basin Block."

APPENDIX, REPORT COMMITTEE P-1, 1928.

METHODS OF TAMPING AND FEEDING IN MANUFACTURE OF DRY TAMPED CONCRETE MASONRY UNITS.

Investigation conducted by Committee P-1.

INTRODUCTION.

This report covers a preliminary investigation of methods of tamping and feeding employed in the manufacture of dry tamped concrete masonry units. Concrete block were selected for these tests as the most representative of tamped concrete units. The tests were conducted under the auspices of this committee, outlined by C. L. Bourne, chairman, and superintended by J. F. Winkler, secretary. Acknowledgment is made to Messrs. Martin and George Hammerschmidt for furnishing the necessary materials used in the manufacture of the block and for use of the Elmhurst-Chicago Stone Company block plant where specimens were made. The specimens were tested at the research laboratory of the Portland Cement Association.

OUTLINE OF TESTS.

The test program as outlined in Table I includes two gradings of aggregates, two consistencies, four methods of feeding concrete into the mold and four numbers of tamps, aggregating twelve variables combined to give 56 different conditions.

Grading of Aggregates.—Group I.—Pea gravel was combined with torpedo sand giving a combined aggregate with fineness modulus of 4.23.

Group II.—Torpedo sand (F.M. 3.23) was used. This sand is coarser than ordinarily used in the manufacture of concrete masonry units.

Consistency.—In these tests the concrete of dry consistency contained sufficient water to produce blocks with slight web markings. The concrete of wet consistency produced blocks with very pronounced, running web marks.

Method of Feeding.—Concrete was fed into the mold in one, two, three and four layers, the last approximating a continuous feed.

Number of Tamps.—The number of total tamps included 6, 12, 18 and 24 strokes of each of four full tamping feet working alternately.

MANUFACTURE OF BLOCK.

The specimens were made at the Elmhurst-Chicago Stone Company block plant under regular operating conditions. The only change in plant personnel was the engineer who proportioned the batches and ran the mixer. Portland cement taken from the same carload at the plant was used.

Sand used in Group I was obtained from the storage bin at the plant. The gravel for this group and the sand used in Group II were purchased from a local dealer. All aggregates were of good quality. The sand

(F.M. 3.74) and gravel (F.M. 5.55) used in Group I were combined in the proportion of 73 per cent and 27 per cent by volume, respectively, giving a combined aggregate with fineness modulus of 4.23.

A mix of 1 cu. ft. of cement to 7.35 cu. ft. of dry rodded aggregate was used. Three days before making the specimens a preliminary sieve analysis was made to determine the quantities of materials required for a 1 : 7 mix. Analysis made during the manufacture of the block showed the mix to be 1 : 7.35.

One-bag batches were made in all cases and the aggregates for each batch were measured in a damp and loose condition in a 1-cu.-ft. box. In

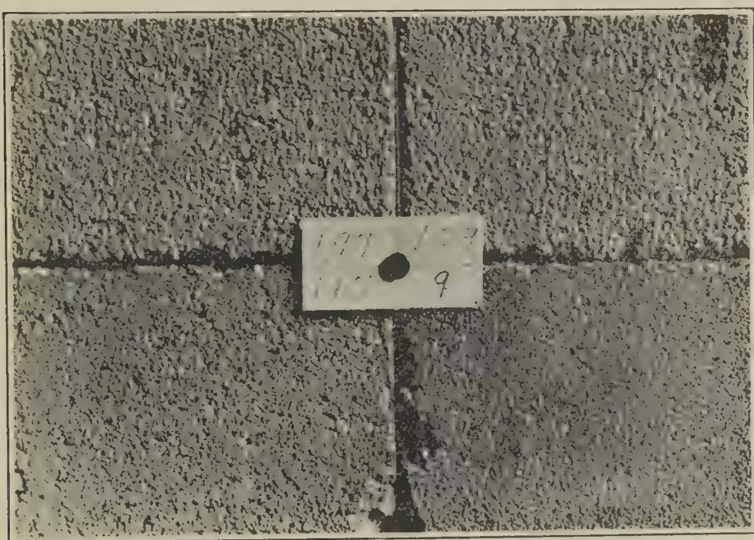


FIG. 3.—THE TEXTURES OF "WET" AND "DRY" CONCRETE BLOCK SPECIMENS WITH FINE AND COARSE AGGREGATES.

197 coarse aggregate, "wet" mix; 117 coarse aggregate, "dry" mix; 109 fine aggregate, "wet" mix; 9 fine aggregate, "dry" mix.

Group I, 6.75 cu. ft. of sand (F.M. 3.74) and 2.33 cu. ft. of gravel (F.M. 5.55) were used in each batch and in Group II, 9 cu. ft. of sand (F.M. 3.23). (See Appendix A for computations to determine the field mix.)

Wet and dry consistencies were used in Groups I and II. The consistency of the trial batch for each condition was judged by three regular plant operators and the engineers in charge. The water required for each batch was weighed on a 50-lb. spring scale and the same quantity used for each condition. For the dry consistency in Group I, 27 lb. of water was added and 40 lb. for the wet consistency. In Group II, 31 lb. and 52 lb. were added.

The concrete for each batch was mixed dry 1½ min. and 3 min. after adding the water in an 11 cu. ft. bottom discharge batch mixer. A pusher

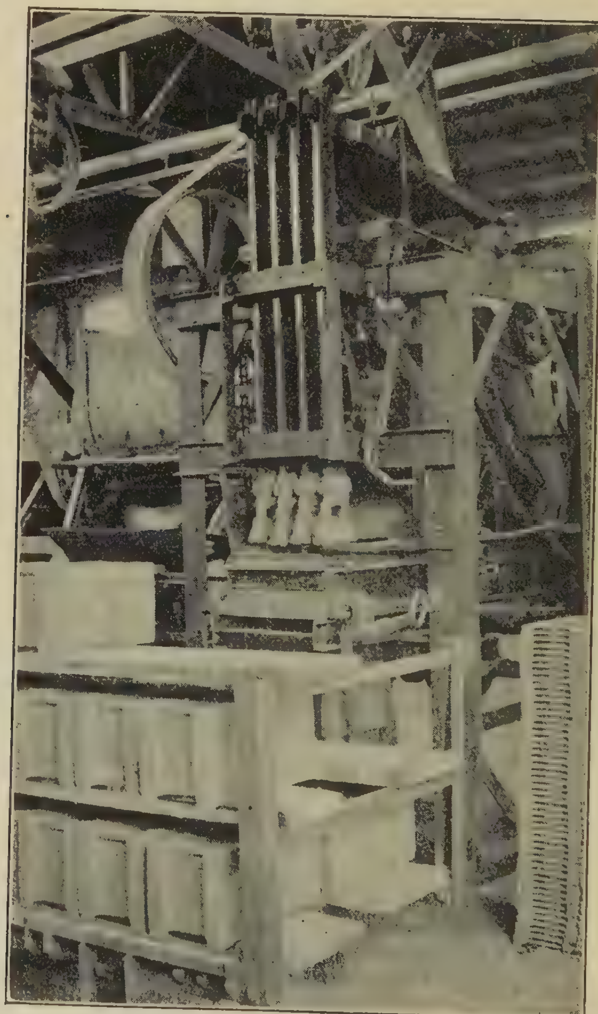


FIG. 1.—VERTICAL STRIPPER TAMPING MACHINE AND MIXER FOR ADJACENT MACHINE IN BACKGROUND.

Test specimens are on rack in the foreground

type of chain conveyor carried the concrete from the mixer hopper to the overhead hopper of the tamping machine except when the concrete was fed into the mold in four layers by shoveling from the floor. The con-

crete was fed into the mold in one, two, three or four layers according to the requirements of the tests. The block machine was a standard vertical stripper with four full-width mechanical tamping feet, making one standard 8 x 8 x 16 block with three oval cores. Each tamper weighed 66½ lb. and dropped 11 in., measuring from the top of the mold to the underside of the foot in a raised position. The machine was run by the regular operator who deviated from ordinary practice only for introducing the variables in the manufacturer's process.

The average size of 15 blocks was 8 x 7¼ x 15¾ in. giving a gross area of 126 sq. in. and net area of 78.9 sq. in.

The total number of tamps on each specimen were equally divided on each layer as nearly as possible. The total number of tamps on each

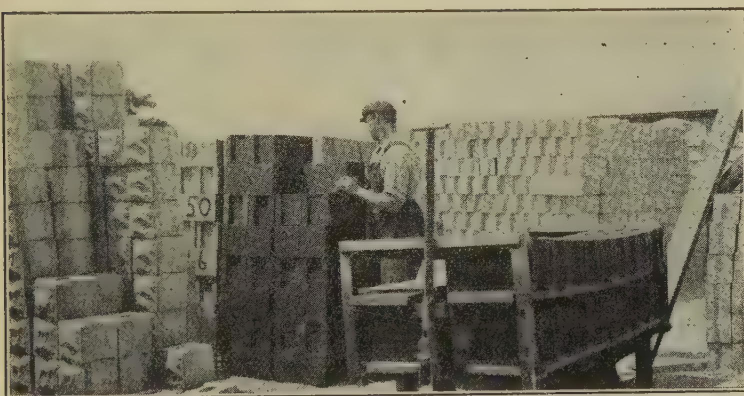


FIG. 2.—PILING TEST BLOCK IN THE STORAGE YARD IMMEDIATELY AFTER STEAM TREATMENT.

specimen was 6, 12, 18 or 24 strokes of each of four full tamping feet working alternately. The blocks were placed on racks and carried to the steam rooms by lift trucks. They were subjected to steam treatment at 100 deg. for 21 hours, then piled eight high in the outside storage yard and after 16 days removed to the laboratory.

TESTING.

Blocks were tested for compressive strength and absorption according to the methods prescribed in the American Concrete Institute Standard Specifications for Concrete Building Block and Tile. Blocks in Group II were not tested for absorption. In making the absorption tests of the blocks in Group I they were dried to constant weight after immersion. Blocks used for the absorption tests were also used for the compression test. Absorption tests were made at 21 days. The compression tests were made at 29 days.

Four specimens were made for each of the 56 conditions. The first block made was discarded and the other three used in this investigation.

DISCUSSION OF RESULTS OF THIS INVESTIGATION UNDER THE CONDITIONS OF TEST.

The conclusions on the effects of different methods of tamping arrived at from these test results may simply be called tendencies, for the actual difference in strengths is not sufficient to warrant clear-cut definite deductions. Results are given in Tables 2 to 6 and in Fig. 1.

FIG. 4.—CENTERING A SPECIMEN BLOCK IN THE 300,000-LB. RIEHLE TESTING MACHINE.

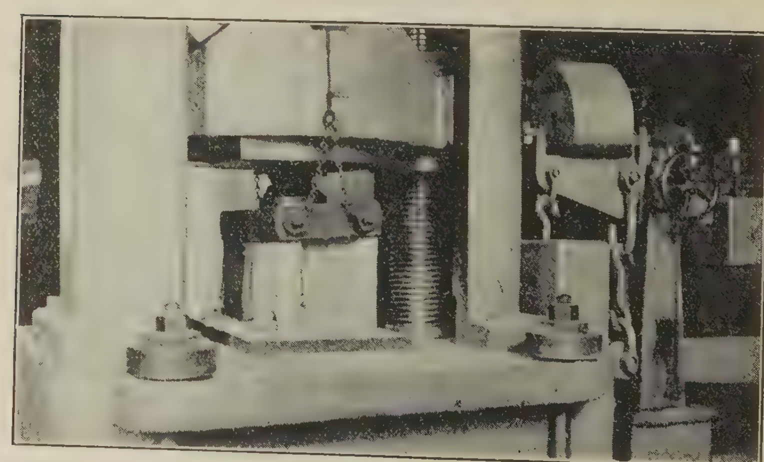


FIG. 5.—COMPLETE FAILURE SHOWING A ONE-SIDED BREAK AND ANOTHER REASON FOR USING A WELL-LUBRICATED SPHERICAL BEARING BLOCK.

(1) In general, specimens made by feeding concrete in one or two layers had approximately the same strength. Those fed in three or four layers also gave about equal strengths.

(2) In the cases of one- or two-layer feeds an increased number of tamps generally resulted in comparatively little increase in strength, while in those of three- or four-layer feeds noticeable increases resulted.

(3) With coarse aggregate, concrete of wet consistency gave higher strength than dry concrete.

(4) With fine aggregate, concrete of wet consistency gave lower strength than dry concrete.

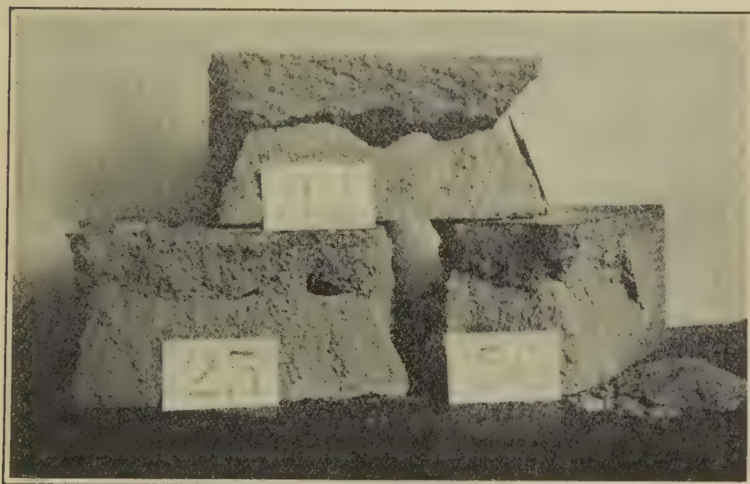


FIG. 6.—TYPICAL FAILURES OF SPECIMENS SHOWING CONICAL SHEAR.

In 1924, Committee P-6 conducted an extensive series of tests, results of which showed "that in general the fundamental laws governing the properties of plastic concrete mixtures may be applied to the drier mixtures used in the commercial manufacture of concrete products." One of the principal factors controlling the quality of concrete is the ratio of water to cement. "For tamped machine-made products the highest strength is obtained when the amount of mixing water used is increased to a quantity sufficient to provide a workable mix."

From the results of these tests it appears that more water than is necessary for maximum strength can be used with a fine aggregate in producing block which will not slump when stripped from the mold.

(5) For the block made with coarse aggregate the highest strengths were obtained in the wet consistency when the concrete was fed into the mold in three layers. In the dry consistency the highest strengths were obtained by feeding the concrete into the mold in two layers. For the block made with sand only as aggregate the highest strengths were ob-

tained when the concrete was fed in three layers. In the latter case this was true for both wet and dry consistencies (see Fig. 2).

(6) Under similar conditions coarse aggregate produced concrete units of higher strength than fine aggregate.

(7) The block made from concrete of wet consistency had an absorption about 2 per cent lower than those from dry concrete.

(8) In general 18 tamps produced the strongest block. For 18 tamps and all conditions the strengths ranged from 676 to 1,290 lb. per sq. in.; for 12 tamps the range was from 599 to 1,227 lb., and for 6 tamps the range was 555 to 1,111 lb. The results of these tests show smaller differences in strengths for the different conditions than was anticipated.

TABLE 1.—OUTLINE OF TESTS

Effects of different methods of tamping and feeding employed in the manufacture of dry tamped concrete masonry units.
Compression and absorption tests of 3 oval core block 8 x 8 x 16.
Mix: 1:7.35 by volume of dry-rodded aggregate.
Time of Mixing 1½ min. dry, 3 min. wet.
Blocks made on a standard vertical stripper single block machine with four alternating tamping feet by the regular plant operator. Nineteen blocks per sack of cement.
Curing: 21 hours in steam at 100° F. Sixteen days outside storage and 12 days room storage.
Age at test: 29 days for compression, 21 days for absorption.
Four specimens made for each condition.

Group and Fineness Modulus	Consistency	Method of Feeding Concrete	Number of Tamps
Group I / Fineness Modulus 4.23	Dry / Wet	1 layer	6, 12, 18, 24
		2 "	6, 12, 18
		3 "	6, 12, 18
		4 "	6, 12, 18, 24
Group II* / Fineness Modulus 3.23	Dry / Wet	1 layer	6, 12, 18, 24
		2 "	6, 12, 18
		3 "	6, 12, 18
		4 "	6, 12, 18, 24

* No absorption tests.

TABLE 2.—SIEVE ANALYSES OF AGGREGATES

Ref. No.	Kind of Aggregate	Per Cent Coarser than Each Sieve—by Volume							Fineness Modulus
		No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜ in.	
1	Fine Torpedo Sand	98.4	90.3	66.2	45.0	20.0	3.0	0.0	3.23
2	Coarse Torpedo Sand	98.0	91.1	73.2	55.2	38.4	18.0	0.0	3.74
3	Roofing Gravel	100.0	99.4	99.0	97.8	92.3	66.1	0.0	5.55
4*	73% of Sand No. 2 combined with 27% of No. 3	98.5	93.4	80.2	66.7	53.1	30.9	0.0	4.23

* Ratio of fine to coarse $= \dfrac{\text{F.M. (coarse)} - \text{F.M. (combined)}}{\text{F.M. (coarse)} - \text{F.M. (fine)}}$

Evidently the strength of concrete block made under similar conditions depends largely on the density of the concrete. It is probable that the impact delivered by the tampers of the machine used was sufficient to produce block under the conditions of test of close to maximum possible density, or that the blows were so light that the maximum possible density could not be reached by continual tamping.

The committee intends further investigations of the effects of variations in the amount of impact by changing the weight of the tampers, the length of stroke or sequence of blows.

TABLE 3.—DATA FOR CALCULATING THE FIELD MIX

Computations for converting the real mix by dry rodded volume to the field mix by damp-loose volume were made from data taken during the proportioning of batches for test specimens. Unit weights determined as prescribed by A.S.T.M. Moisture content per cent by weight of dry aggregate.

Ref. No. of Agg.	Weight of Sample		Moisture, per cent by Weight	Shrinkage, per cent	Unit Weight, lb. per cu. ft.		Weight of Dry Aggregate in 1 cu. ft., Damp Loose	No. of cu. ft. Damp Loose to Give 1 cu. ft. Dry Rodded	Bulking, per cent
	Wet	Dry			Dry Rodded	Damp Loose			
1	9.50	9.70	3.28	....	112.5	95.0	92.0	1.225	22.5
2	9.75	9.30	4.84	....	112.0	97.5	93.0	1.205	20.5
3	9.83	9.55	3.04	....	108.0	98.4	95.5	1.130	13.0
4	....	....	....	96.0	116.0	....	....	.....	....

TABLE 4.—COMPRESSION TESTS OF BLOCK IN GROUP I

Compression tests of 3 oval core block 8 x 7¾ x 15¾ in.
Mix: 1:7.35 dry rodded volume.
Aggregate: 73% torpedo sand, 27% pea gravel.
Consistency: Dry 7.7 gal. per sack, wet 9.3 gal. per sack.
Time of mixing: 1½ min. dry, 3 min. wet.
Curing: 21 hours in steam at 100° F., 16 days outside storage, 12 days room storage.
Age at test, 29 days.
Blocks made by regular plant operator on a standard vertical stripper single block machine with 4 alternating tamping feet.

Method of Feeding	Compressive Strength, lb. per sq. in.									
	Dry					Wet				
	Number of Tamps					Number of Tamps				
	6	12	18	24	Grand Average	6	12	18	24	Grand Average
1 layer	905	833	759	896		952	1035	1122	1032	
	676	714	835	925		860	860	860	927	
	698	760	770	865		1013	1098	1001	870	
Av.	760	769	788	895†	772	941	998	994	943†	978
2 layers	1058	976	1124	....		1120	1070	1184	....	
	998	989	1202	....		997	963	1180	....	
	....	....	1259	....		....	....	1131	....	
	1021	1218	923	....		928	999	1263	....	
Av.	1026	1061	1127	....	1071	1015	1011	1190	....	1072
3 layers	842	1130	953	....		1131	1254	1196	....	
	843	1092	742*	....		1060	1201	1328	....	
	970	966	....	....		1143	1217	1138	....	
	924	706*	957	....						
Av.	895	1063	955	....	971	1111	1227	1221	....	1183
4 layers	747	727	968	1070		1062	1170	1235	1171	
	626	744	837	1118		1000	1222	1336	1460	
	729	....	....	....		1073	1270	1298	1288	
	662	751	1012	1011						
Av.	691	741	939	1066†	780	1052	1221	1290	1360†	1188
Grand Average	843	909	952	980		1030	1112	1174	1125	

* Defective specimen omitted from average.
† Not included in Grand Average for 1 layer and 4 layers.

TABLE 5.—COMPRESSION TESTS OF BLOCK IN GROUP II

Compression tests of 3 oval core block 8 x 7¾ x 15¾ in.
Mix: 1:7.35 dry rodded volume.
Aggregate: Torpedo sand, F. M., 3.23.
Consistency: Dry 7 gal. per sack, wet 9.5 gal. per sack.
Time of mixing: 1½ min. dry, 3 min. wet.
Curing: 21 hours in steam at 100° F., 16 days outside storage, 12 days room storage.
Age at test, 29 days.
Blocks made by regular plant operator on standard vertical stripper single block machine with 4 alternating tamping feet.

Method of Feeding	Compressive Strength, lb. per sq. in.									
	Dry					Wet				
	Number of Tamps					Number of Tamps				
	6	12	18	24	Grand Average	6	12	18	24	Grand Average
1 layer..........	793	709	728	887		627	520	780	600	
	643	788	856	737		544	677	647	569	
	675	785	632*	690		503	482*	721	498*	
	Av. 704	761	792	812	752	Av. 555	599	716	585†	623
2 layers.........	744	743	757	...		700	611	650	...	
	694	720	927	...		565	613	696	...	
	*	681	704	...		587	589	690	...	
	Av. 719	715	796	...	743	Av. 617	604	679	...	633
3 layers.........	775	854	779	...		570	730	712	...	
	747	956	972	...		648	724	818	...	
	688	820	852	...		576	770	785	...	
	Av. 735	877	868	...	827	Av. 598	741	772	...	704
4 layers.........	647	908	950	933		642	708	652	728	
	793	808	1035	829		651	716	689	729	
	812	752	999	863		650	606	687	702	
	Av. 751	823	995	875†	856	Av. 648	677	676	720†	667
Grand Average..	727	794	863	844		605	655	711	653	

* Defective specimen omitted from average.
† Not included in Grand Average for 1 layer and 4 layers.

TABLE 6.—ABSORPTION TESTS OF BLOCK IN GROUP I

Blocks Used for Compression Test

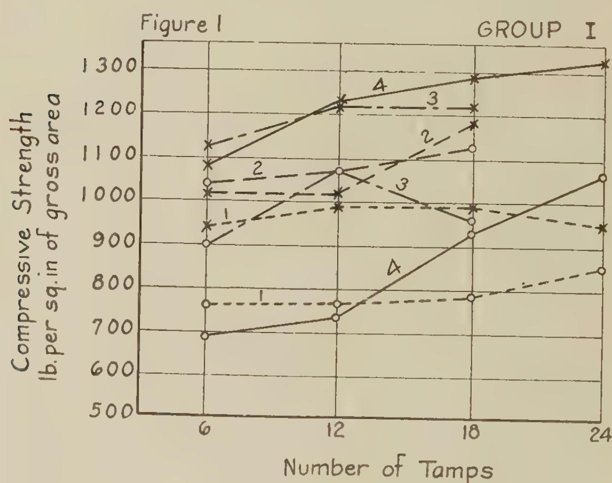
Method of Feeding	Absorption, per cent by weight of dry block									
	Dry					Wet				
	Number of Tamps					Number of Tamps				
	6	12	18	24	Grand Average	6	12	18	24	Grand Average
1 layer..........	8.5	8.8	9.2	8.3		7.4	4.9	6.3	5.7	
	8.9	8.4	9.5	8.0		5.6	6.5	7.3	7.0	
	8.4	8.1	8.0	9.3		6.2	5.6	6.4	7.1	
	Av. 8.6	8.4	8.9	8.5*	8.6	Av. 6.4	5.7	6.7	6.6*	6.3
2 layers.........	8.1	7.3	7.8	...		7.1	7.0	7.9	...	
	8.5	7.2	6.4	...		7.5	7.2	6.7	...	
	7.6	7.1	7.7	...		6.0	7.2	6.2	...	
	Av. 8.1	7.2	7.3	...	7.5	Av. 6.9	7.1	6.9	...	7.0
3 layers.........	8.0	7.4	8.1	...		6.6	7.1	7.1	...	
	7.2	7.2	7.7	...		7.5	6.0	6.6	...	
	8.3	8.2	8.3	...		7.0	6.9	6.6	...	
	Av. 7.8	7.6	8.0	...	7.8	Av. 7.0	6.7	6.8	...	6.8
4 layers.........	9.3	7.0	5.4	7.2		6.1	6.5	6.2	6.5	
	9.4	8.6	6.0	5.2		5.7	5.6	6.6	6.1	
	9.0	7.8	7.4	6.4		6.6	6.2	5.3	5.5	
	Av. 9.2	7.8	6.3	6.3*	7.8	Av. 6.1	6.1	6.0	5.4*	6.1
Grand Average..	8.4	7.8	7.6	...		6.6	6.4	6.6	...	

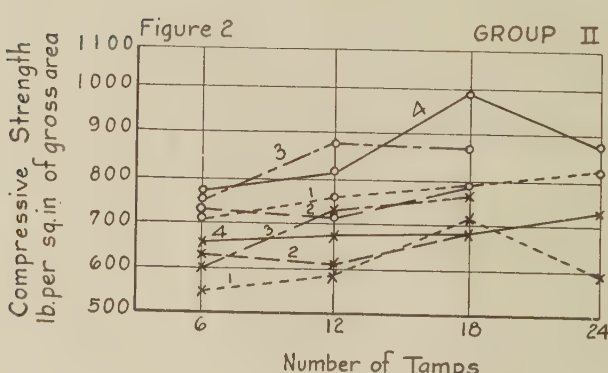
* Omitted from Grand Average for 1 layer and 4 layers.

GRAPH SHOWING RELATION BETWEEN COMPRESSIVE STRENGTH AND NUMBER OF TAMPS FOR FOUR DIFFERENT METHODS OF FEEDING CONCRETE

o Represents Dry concrete
x " Wet concrete

NOTE: Figures on lines indicate number of layers of feed.





APPENDIX A.

Calculating the Field Mix.

Group I.—Calculations on sample aggregate analyzed before blocks were made are as follows:

1 : 7 mix by dry rodded volume

$1 : \frac{7}{.9}$ to account for shrinkage of 90 per cent

1 : 7.8 result

1 : 7.8 x 0.73 — 7.8 x .27 73 per cent fine aggregate and 27 per cent coarse aggregate

1 : 5.7 — 2.1 result

1 : 5.7 x 1.18 — 2.1 x 1.11 to account for bulking

1 : 6.75 — 2.33 field mix actually used

With a change in the aggregates the following computations give the actual real mix used. See Table 3.

1 : 6.75 — 2.33

$1 : \frac{6.75 — 2.33}{1.205 \quad 1.13}$ to account for bulking

1 : 5.58 — 2.06 result

$1 : \frac{5.58}{.73} — \frac{2.06}{.27}$ 73 per cent fine aggregate and 27 per cent coarse aggregate

1 : 7.65 result

1 : 7.65 x 0.96 to account for shrinkage of 96 per cent

1 : 7.35 mix by dry rodded volume

Group II.—The per cent bulking for the sand used in making the specimens in Group II was 22.5 per cent. Using the same dry rodded mix as in Group 1 the resulting field mix was 1 : 7.35 x 1.225 = 1 : 9.

APPENDIX B.

WATER-CEMENT RATIO CALCULATIONS.

Group I.—Real mix 1: 7.35; field mix 1: 6.75 — 2.33; weight of dry aggregate in 1 cu. ft. damp and loose times per cent moisture = weight of water in 1 cu. ft. damp loose.

Aggregate No. 2—93.0 x .0484 = 4.5 x 6.75 = 30.38
Aggregate No. 3—95.5 x .0304 = 2.9 x 2.33 = 6.76

37.14
Water added for *dry consistency* = 27.00

64.14 lb.
64.14 ÷ 8.33 = *7.70 gal. per sack of cement*
37.14
Water added for *wet consistency* = 40.00

77.14
77.14 ÷ 8.33 = *9.3 gal. per sack of cement*

Group II.—Real mix 1: 7.35; field mix 1: 9.
Aggregate No. 1—92.0 x .0328 = 3.02 x 9 = 27.18
Water added for *dry consistency* = 31.00

58.18
58.18 ÷ 8.33 = *7.00 gal. per sack of cement*
27.18
Water added for *wet consistency* = 52.00

79.18
79.18 ÷ 8.33 = *9.5 gal. per sack of cement*

Absorption of aggregate is neglected because samples were sun-dried. Samples were spread on sheet metal and exposed to sun light until dry enough for the sieve analysis.

Discussion—Standard Building Units.

C. L. Bourne.—In one or two cases the preprint stated that the committee had not approved of several of the sections. That was before all the votes were in, and at present time the report has been approved by the committee with the exception of the paragraph dealing with the specifications on concrete brick. There are two fairly important things in this report, the possible advancement from tentative standards to standard of the specifications on concrete brick and the specifications on concrete manhole and sewer block. Therefore, in presenting the report for the approval of the convention, I suggest that the report be considered in three sections, that is, that all of the report be considered except the two specifications mentioned, and that these two specifications be considered separately. If it is not out of order for me to do so, I would like to move that the report of Committee P-1 be accepted with the exception of the specifications on concrete brick and with the exception of the sections on concrete manholes and catch basin block. Mr. Bourne.

Motion seconded and unanimously adopted.*

The committee reported against the advancement at the present time of the specifications for concrete brick from a tentative to a standard specification. The principal reason for holding up this particular specification longer was due to a discussion by Mr. Chapman in which he called attention to the fact that there are two organizations, both of them making specifications for concrete brick, that they differ in some respects, but that the differences are not irreconcilable. He suggested that the committee of the American Concrete Institute get together with the committee of the American Society for Testing Materials and see whether or not the specifications cannot be made more uniform.

C. M. Chapman.—From my talk on the subject with those most interested, I do not think there is any difference of opinion, and therefore no necessity for taking up the time of the meeting to convince people who are already convinced. If there is any comment to the contrary, then I would be glad to talk a little while or a long time on that subject. Mr. Chapman.

C. L. Bourne.—On account of this situation, I move that the tentative standard specification for concrete brick be continued for another year as a tentative specification. Motion seconded. Mr. Bourne.

S. H. Ingberg.—In connection with the attempt to unify the specifications, I might call attention to a forthcoming specification by the Federal Mr. Ingberg.

* The effect of the convention action is to make a revision in the Institute's "Tentative Specifications for Concrete Block and Concrete Building Tile"—P1-A-26T, which therefore remains tentative as serial P1-A-28T.

Specifications Board on Concrete Brick. It was under consideration a few years ago, but was laid aside and taken up again only a few weeks ago. This session is not devoted to a discussion of the detailed requirements for concrete brick, so I will not go into that part of it, but I will only call attention to the differences between the proposed federal specification and the specification of this Institute.

When we began work on specifications for brick 4 or 5 years ago in the Federal Specifications Board, we were met by the demand from the representatives of the government departments, that whatever requirements were made for brick, they must be made such that the acceptance can be made with the greatest possible convenience and expedition. They did not wish to be tied up to a procedure that might delay important work, and federal officers are often not in a position to waive the requirements. The first proposition was to base everything on absorption. This has been used as a rough measure of the properties of clay brick, but in the case of sand-lime brick and concrete brick, that test is obviously not adequate. The other test that appeared possible to make was the transverse breaking test. This test is more simple than the compression test and it can be adapted for field and plant inspection, hence the specifications for clay brick and sand-lime brick came through with an absorption requirement for the respective classes of brick. We appreciated, in the case of clay brick, that the relation between the transverse strength of brick and the compressive strength is not well defined, and that possibly the strength of the masonry is somewhat better defined by the compressive strength of individual bricks than by the transverse strength. We believed, however, considering that the transverse strength of brick does have some relation to the strength of the masonry, that for the purpose of the specifications the transverse test taken in conjunction with an absorption test will grade the bricks into the three or four classes with a fair degree of adequacy. Of course, in the case of sand-lime brick, the relation between the compressive strength and the transverse strength is better defined than for clay brick, so that for this type of brick the specifications are on a little better basis than for clay brick. For concrete brick also, we found by examining what data there is, that the relation between the compressive strength flatwise and the transverse strength is fairly well defined, hence, even if we consider the compressive strength of the units to be the stress criterion of the measure of the strength of the masonry, the transverse strength measures the compressive strength proportionately to such extent that it gives practically a near equivalent.

We also have in the proposed concrete brick specifications a maximum absorption requirement. We found that the relation between strength and absorption for concrete brick is very poorly defined, as it depends on the grading of the aggregates as well as on the mix, so that we simply placed a maximum absorption limit for each class, basing the classification mainly on the transverse strength. I hope this will make clear that the provisions of the given specifications are what they are, not because of a desire to do

something different, but because this particular procedure suited the conditions for which the specifications were formulated.

The motion was unanimously adopted.

C. L. BOURNE.—The committee recommends that the standard specification for concrete manhole and catch basin block be advanced from tentative specification to standard specification, but before putting a motion to that effect, I would like to see if there is any discussion from the floor. **Mr. Bourne.**

V. D. M. ALLAN—In view of Mr. Christensen's remarks and considerable information that has come to light since the committee took action and voted favorably on advancing the tentative specifications of manhole block, it seems to me that we should in the title of the specifications limit the material to exactly the use that I believe was intended when the specification was first originated. As the title now reads, I believe it is tentative specifications for concrete manhole and catch basin block, and that would cover the use of manholes and catch basin block in all kinds of construction. I believe that in the beginning it was intended that the specification should cover manholes and catch basin block in sewer construction. There are a number of uses of manhole block in other than sewer construction, where the same specifications are not necessary. I would like to suggest that the word sewer be inserted in the title, to read Tentative Specifications for Concrete Sewer Manhole and Catch Basin Block. **Mr. Allan.**

EINAR CHRISTENSEN.—I am familiar with most of the data referred to, and I would like to second that suggestion, because I think that it will do an injustice to quite a few manufacturers to adopt the specifications with the present title. **Mr. Christensen.**

C. L. BOURNE.—I would like to move that the present tentative specifications for concrete manhole and catch basin block be continued as tentative for another year and that a tentative amendment be made to these specifications so that the title shall read Standard Specifications for Concrete Sewer Manhole and Catch Basin Block. **Mr. Bourne.**

Motion seconded and unanimously adopted.

Report of Committee S-6 on Concrete Roads and Pavements.

At the Institute convention, Feb. 26, 1926, a motion was adopted on recommendation of this committee, to revise Standard Specifications for One-Course Portland Cement Concrete for Highways in Paragraph 5b, p. 696, Vol. 20, A. C. I. *Proceedings*, excepting slag from the wear test requirement so that the last sentence will read: "Coarse aggregate, excepting air-cooled furnace slag, shall show not more than 6 per cent loss in the wear test"; and omitting from the "Note" (same page) the sentence, "As a guide to the engineer, Abrams' 'Tables of Proportions and Quantities for Concrete Road Construction' are printed herewith."

Since that time there have been no changes in the revisions as submitted and the committee recommends that this tentative revision be submitted to a vote of the membership of the Institute as a standard for "Standard Specifications for One-Course Portland Cement Concrete for Highways, S-6A-24 and for three other standards of which it is a part; Two-Course Portland Cement Concrete Pavement for Highways, S-6B-25; One-Course Portland Cement Concrete Street Pavement, S-6C-25; and Two-Course Portland Cement Concrete Street Pavement, S-6D-25.

The attention of the committee has been brought to the possible ambiguity of a portion of the wording in Part II, Materials, Sec. (E) Joint Filler, Paragraph 18 of the Specifications for One-Course Concrete Pavements for Highways, S-6A-24. To remove this ambiguity the committee approves the following revisions:

In the second line, after "bitumen," add "or a uniform mixture of fibre and bitumen, or a combination of both." Also in the second line, after "per cent" add, "by weight."

Paragraph (e) Joint Filler, as revised, will therefore read:

"Joint fillers shall consist of prepared strips of fibre matrix and bitumen, or a uniform mixture of fibre and bitumen, or a combination of both, containing not more than 25 per cent by weight of inert material, having thickness of —— inches, and width equal to —— inches greater than the thickness of the pavement at any point. The bitumen used in the manufacture of the joint filler may be either tar or asphalt of a grade that will not become soft enough to flow in hot weather, nor brittle in cold weather. The prepared strips shall be cut to conform to the cross-section of the pavement and in lengths equal to the width of the pavement, except that strips equal in length to half the width of the pavement may be used when laced or clipped together at the center in a workmanlike and effective manner."

The committee recommends that this change be submitted to the Institute for adoption as tentative standard in the Specifications S-6A-24, S-6B-25, S-6C-25 and S-6D-25.*

This report has been submitted to letter ballot of the committee which consists of 6 members, of whom 5 have voted affirmatively, 0 negatively, and 1 has refrained from voting.

COL. W. M. ACHESON, *Chairman.*
L. S. TRAINOR, *Secretary.*

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* These changes were adopted tentatively by the 1928 convention.

** The following two paragraphs appeared on a letter ballot canvassed June 1, 1928, which resulted in the adoption of the tentative revisions of 1925, so that road standards S-6A-24, S-6B-25, S-6C-25 and S-6D-25 are changed to S-6A-28, S-6B-28, S-6C-28 and S-6D-28: At the 1925 convention, revisions were adopted tentatively to Standard Specifications for One-Course Portland Cement Concrete for Highways in Paragraph 5b, p. 696, Vol. 20, A. C. I. *Proceedings,* excepting slag from the wear test requirement so that the last sentence will read: "Coarse aggregate, excepting air-cooled furnace slag, shall show not more than 6 per cent loss in the wear test"; and omitting from the "Note" (same page) the sentence, "As a guide to the engineer, Abrams' 'Tables of Proportions and Quantities for Concrete Road Construction' are printed herewith."

At the 1928 convention the standards thus tentatively revised were referred to letter ballot. The tentative revision in the "Standard Specifications for One-Course Portland Cement Concrete Pavement for Highway, S-6A-24", affects three other standards of which it is a part; Two-Course Portland Cement Concrete Pavement for Highways, S-6B-25; One-Course Portland Cement Concrete Street Pavement, S-6C-25; and Two-Course Portland Cement Concrete Street Pavement, S-6D-25.

AMERICAN CONCRETE INSTITUTE

BUSINESS REPORTS

Annual Report of the Board of Direction to the Members—May 1, 1928.

Since this annual volume of *Proceedings* is in itself a comprehensive report of the Institute's technical work, this report aims only to set forth information in relation to membership and financial condition.

The annual audit made by the Meissner Audit Co., Detroit, for the period July 1, 1926, to June 30, 1927—the latest fiscal year period—shows what funds have been received and disbursed and an increase of surplus from the previous fiscal year. As the membership increases there comes an increased responsibility to "carry on," with the accompanying desirability of such small annual additions to surplus as might tide the Institute over a period of less active growth, should that time come.

In the fiscal year ended June 30, 1927, eight News Letters, four of 8 pages each and four of 16 pages each, were issued. Thus far in the present fiscal year, July 1, 1927, to May 1, 1928, there have been six News Letters, totaling 72 pages together, with a 100-page Directory.

Nineteen preprints including 22 papers and reports were issued to members, previous to the 1928 convention.

The registration at the Philadelphia 1928 meeting was 700—an increase of 50 over the 1927 meeting in Chicago.

The recent increase in membership has been slightly less rapid, possibly due to an increase in dues from $10.00 to $12.50 from July 1, 1927. This increase, voted overwhelmingly by the members on letter ballot, puts the organization in a very much stronger position—able to continue its enlarged program of publications, through which the Institute must continue to expand its service.

In the fiscal year ended June 30, 1927, 2 honorary members were elected; 579 new active members were added and 205 were lost, a net gain of 374. In that period 17 supporting members were added; 13 were lost, a net gain of 4. The active membership at the close of the fiscal year was 2,193, supporting membership 120, honorary members 2, total at the close of the fiscal year 2,315.

Since then to April 1, 1928, further gains bring the active membership to 2,478, supporting membership 123, honorary membership 2—total 2,603, a net gain of 288 members in the first nine months of the present fiscal year.

Following is a report of the annual audit for the fiscal year ended June 30, 1927:

MEISSNER AUDIT CO.

DETROIT, MICH.

August 4, 1927.

Mr. Harvey Whipple, Treasurer,
American Concrete Institute,
2970 W. Grand Boulevard,
Detroit, Michigan.

Dear Sir:

We have conducted a cash audit of the American Concrete Institute, from July 1, 1926, to June 30, 1927, as shown by the books of the company, and have found that all cash reported as received has been deposited with the National Bank of Commerce, Detroit, Michigan. Disbursements and withdrawals are covered by proper voucher checks, which are all approved.

The bank account was verified, and agrees with statement of the National Bank of Commerce at June 30, 1927.

No attempt has been made to verify cash reported as received, with the various members.

The following Schedules are attached hereto: Balance Sheet as at June 30, 1927; Receipts and Disbursements, July 1, 1926, to June 30, 1927; Bank Reconciliation, June 30, 1927.

Thanking you for the assistance rendered and courtesy shown, during the conduct of our audit, we remain,

Yours respectfully,

Meissner Audit Company,

By John C. Meissner,

Public Accountant.

AMERICAN CONCRETE INSTITUTE.

BALANCE SHEET.

June 30, 1927.

ASSETS.

Cash:		
On hand	$500.00	
In National Bank of Commerce	4,241.04	
Total Cash		$4,741.04
U. S. Treasury Certificates		8,131.25
Accounts Receivable:		
Active Members	$3,170.00	
Contributing Members	450.00	
Miscellaneous	1,223.15	
Total Accounts Receivable		4,843.15
Inventories:		
177—1905-1919—@ $0.50	$88.50	
756—1920-1926—@ $1.00	756.00	
215—1927—@ $3.00	645.00	
Total Inventories		1,489.50
Total Assets		$19,204.94

LIABILITIES.

Accounts Payable—Proceedings	$5,991.95	
Deferred:		
Dues Paid in Advance	219.00	
Reserve:		
For Loss Due to Delinquent Members	3,500.00	
Surplus:		
Deposited by Members	9,493.99	
Total Liabilities		$19,204.94

AMERICAN CONCRETE INSTITUTE.

RECEIPTS AND DISBURSEMENTS.

Year Ending June 30, 1927.

Cash on hand July 1, 1926		$5,719.46
RECEIPTS.		
Dues, Active	$20,314.03	
Dues, Contributing	5,800.00	
Preprint Sales	1,072.53	
Proceeding Sales	1,851.65	
Interest Earned—Bank Balances	51.49	
" " —Treasury Certificates	272.75	
Certificates (Membership A. C. I.)	5.52	
U. S. Treasury Certificates	2,500.00	
Exchange Items, Net	5.54	
Total Receipts		31,873.51
Total		$37,592.97
DISBURSEMENTS.		
U. S. Treasury Certificates	$6,131.25	
Auditing	25.00	
Binders	12.36	
Conventions	1,038.79	
Express and Freight	85.95	
Membership—National Fire Protection Association	60.00	
Miscellaneous Contingencies	42.40	
Office Expense	583.80	
Postage	1,350.70	
Preprint Expense	2,606.81	
Printing, Multigraphing, Stationery and News Letters	3,423.03	
Proceedings Expense	7,299.47	
Rent	720.00	
Secretary's Bond	50.00	
Salaries	9,466.27	
Traveling Expense	524.82	
Total	$33,420.65	
Less Discounts	68.72	
Total Disbursements		33,351.93
Cash in Bank June 30, 1927		$4,241.04

AMERICAN CONCRETE INSTITUTE.

BANK RECONCILIATION.

June 30, 1927.

Balance—as per Ledger	$4,241.04
Total Outstanding Checks	125.22
Balance per Bank Statement, June 30, 1927	$4,366.26

LIST OF REGISTRANTS.

List of Registrants at 1928 Convention.

Asterisk (*) denotes a member.

*Abel, Norman, Allegheny County D. P. W., 1007 Milton Ave., Swissvale, Pa.
*Abrams, Duff A., 342 Madison Ave., New York, N. Y.
Adams, William, 5319 Pentridge St., Philadelphia, Pa.
*Ahearn, V. P., 837 Munsey Bldg., Washington, D. C.
Artley, L. C., Wm. H. Gravell, Inc., 225 S. 15th St., Philadelphia, Pa.
*Ahlers, John G., 110 W. 40th St., New York, N. Y.
Allan, W. D. M., 33 West Grand Ave., Portland Cement Assn., Chicago, Ill.
Allen, H. B., General Crushed Stone Co., 1019 N. American Bldg., Philadelphia, Pa.
*Allen, L. H., Hawthorne Roofing Tile Co., 507 5th Ave., New York, N. Y.
*Allen, Roscoe, Economy Concrete Co., New Haven, Conn.
*Alvord, Henry B., Northeastern University, 316 Huntington Ave., Boston, Mass.
*Anderegg, F. O., Mellon Institute, Pittsburgh, Pa.
*Anderson, W. P., Ferro Concrete Constr. Co., Cincinnati, Ohio.
*Anderson, Louis, Alpha Portland Cement Co., Easton, Pa.
*Andrews, L. E., 4 Morris Circle, Trenton, N. J.
Antes, D. Edward, City Hall, Coatesville, Pa.
*Austin Co., The, 3765 Highland Ave., Drexel Hill, Pa. (E. F. Archibald.)
*Arnold Stone Co., P. O. Box 3039, Jacksonville, Fla. (M. A. Arnold.)
*Ashton, Frank, Young Bldg., Allentown, Pa.
*Atwater, R. W., 68 Trinity Pl., New York, N. Y.
*Backus, Richard A., Voorhees, Gemlin & Walker, 101 Park Ave., New York N. Y.
*Baker, Samuel, 1521 Quincy Ave., Scranton, Pa.
*Baldwin, F. G., Baldwin-Tarvin Co., 615 Carr St., Cincinnati, Ohio.
Ballinger, Robert, 12th & Chestnut Sts., Philadelphia, Pa.
*Bamman, F. C., Washington, D. C.
Banta, Edwin, 4928 Chestnut St., Philadelphia, Pa.
Baranzelli, Prof., Baranzelli Stone Co., Woodside, N. Y.
*Barr, Charles L., 902 Gidden Lane Bldg., Shreveport, La.
*Barrett, R. L., Westinghouse, 517 Holmes St., Wilkinsburg, Pa.
*Bartlett, I. L., The Art Stone Co., Miller Falls, Mass.
Bartlett, Raymond W., West Creek, N. J.
*Bates, P. H., Bureau of Standards, Washington, D. C.
*Bauer, E. E., 304 W. Iowa St., Urbana, Ill.
*Bartram, Geo. C., 262 Solvit St., Buffalo, N. Y.
Becker, Chas. W., Mohawk Carpet Mills, Inc., Amsterdam, N. Y.
Beckett, Chas. A., The Ballinger Co., Philadelphia, Pa.
*Beggs, Geo. E., Princeton, N. J.
Beckay, Andrie W., Barnay Shlers Const. Co., 110 W. 40th St., New York, N. Y.
Behny, J. Frank, Sinking Spring Concrete Block, 254 South St., Lebanon, Pa.

BELLUCCI, F., Universal Marble Prod. Corp., Pelham Manor, N. Y.
BENNETT, VICTOR T., E. I. duPont de Nemours, Wilmington, Del.
*BENSON, NEWTON D., 36 Burrington St., Providence, R. I.
*BENT, STEDMAN, Raymond Concrete Pile Co., Philadelphia, Pa.
*BERCHAM, R. O., St. Paul Cement Works, St. Paul, Minn.
BERGHOLM, A. O., 246 Academy St., Jersey City, N. J.
*BERNIER, N. M., 411 Walden St., Cambridge, Mass.
*BERTIN, R. L., White Construction Co., Inc., 95 Madison St., New York, N. Y.
*BREWER, B. R., Pottsville Building Block Co., Pottsville, Pa.
*BIGLER, H. P., Rail Steel Bar Assn., Builders' Bldg., Chicago, Ill.
*BILLNER, K. P., The Aerocrete Corp., 51 East 42nd St., New York, N. Y.
*BIRD, MILLARD F., A. C Horn Sales Corp., 101 Park Ave., New York, N. Y.
*BISCHOFF, J. M., 5915 Canton Ave., Detroit, Mich.
BISHOP, GEO. N., Western Waterproofing Co., 1446 Rankin Bl., St. Louis, Mo.
*BISHOP, HOWARD, Sharon Steel Hoop Co., Sharon, Pa.
*BISSELL, CLINTON T., 85 John St., New York, N. Y.
BLACK, W. T., Fairmont Wall Plaster Co., Fairmont, W. Va.
BLAINE, ETHEL E., American Concrete Institute, Detroit, Mich.
BLANCATO, V., 1218 Oak Point Ave., Bronx, N. Y.
*BLANCHARD, F. E., 2706 Terrace Ave., Columbus, Ohio.
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*BOASE, A. J., care of Penn. Military College, Chester, Pa.
BOCKIUS, LOGAN, Like Real Stone Co., Ambler, Pa.
*BOGUE, R. H., Bureau of Standards, Washington, D. C.
BOONIN, M. M., 23 S. 16th St., Philadelphia, Pa.
*BORCHARD, E. K., Atlas Portland Cement Co., 25 Broadway, New York, N. Y.
BORIE, RENSHAW, W. H. Gravell, Inc., 1520 Spruce St., Philadelphia, Pa.
*BOURNE, C. L., 1502 Greenwood Ave., Jackson, Mich.
BOWEN, WALTER PAUL, 3925 Pine St., Philadelphia, Pa.
*BOYD, D. KNICKERBACHER, 112 S. 16th St., Philadelphia, Pa.
BOYD, H. C., Truscon Steel Co., Upper Darby, Pa.
BOYD, PAUL L., 6943 Limekiln Pike, Philadelphia, Pa.
*BOYER, E. D., Atlas Portland Cement Co., New York, N. Y.
BRACKIN, S. H., Pennsylvania R. R., Andalusia, Pa.
*BRADLEY, C. M., Youngstown Sheet & Tube Co., Gary, Ind.
*BRAGG, J. G., Alpha Portland Cement Co., 213 Kensington Ave., Trenton, N. J.
*BRAGGER, E. Y., The Sandusky Cement Co., Cleveland, Ohio.
BRANDIS, R. L., Barney-Ahlers Const. Corp., Newark, N. J.
BRENNAN, WM. E., Edison Portland Cement Co., Philadelphia, Pa.
*BRICKETT, EDW. M., Lehigh Portland Cement Co., Allentown, Pa.
BROANDER, E., Aerocrete Corp. of America, 51 E. 42nd St., New York, N. Y.
*BROOKE, HOWARD, Bergen Building Block Co., Ridgefield Park, N. Y.
BROOKMAN, LOUIS, 139 N. Clark St., Chicago, Ill.
BROOKS, C. F., Portland Cement Assn., 1315 Walnut St., Philadelphia, Pa.
BROWN, H. W., Lawrence Cement Co., New York, N. Y.
*BROWN, H. WHITTEMORE, Concord, Mass.
BROWN, RICHARD A., Lehigh Portland Cement Co., Allentown, Pa.
BRUNER, BERNARD L., California Stucco Products Co., Cincinnati, Ohio.
*BUCKINGHAM, F., Riehle Bros., Testing Machine Co., 1424 N. 9th St., Philadelphia, Pa.
*BUENTE, C. F., Concrete Products Co. of America, Pittsburgh, Pa.
*BULLEN, C. A., 111 W. Washington St., Chicago, Ill.
*BUTLER, S. FRANK, Giant Portland Cement Co., East Aurora, N. Y.
BUTTON, W. G., City of Philadelphia, Philadelphia, Pa.
*CALDWELL, F. E., 99 Cortland St., Springfield, Mass.
CALLAN, MART P., 4144 Aspen St., Philadelphia, Pa.

CALVERT, L. L., 131 Poplar Ave., Wayne, Pa.
CAMP, C. L., Anthracite Bridge Co., Scranton, Pa.
*CAMPBELL, R. H., 25 Broadway, New York, N. Y.
*CAMPBELL, S. A., Ridge Constr. Corp., Rochester, N. Y.
*CAMPION, H. T., 112 S. 16th St., Philadelphia, Pa.
*CANON, F. EUGENE, 721 Ryder Ave., Brooklyn, N. Y.
CANTONE, JOSEPH, 320 Salem St., Wakefield, Mass.
CANTWELL, JOHN J., Southern Brick & Tile Co., Charlotte, N. C.
*CAPOUCH, M. E., 208 S. LaSalle St., Chicago, Ill.
*CARREL, F. G., Calumet Steel Co., Chicago, Ill.
*CARSWELL, J. B., Buffalo Steel Co., Tonawanda, N. Y.
CARTER, S. C., The Koppers Co., Pittsburgh, Pa.
*CARTY, BRUCE F., The Maul Co., 1640 E. Hancock Ave., Detroit, Mich.
CAULK, ARTHUR N., St. Michaels, Maryland.
CAULK, ALVAN, St. Michaels, Maryland.
*CAVANAGH, R. T., John J. Turner & Sons, Amsterdam, N. Y.
*CHAPMAN, CLOYD M., 105 W. 40th St., New York, N. Y.
CHALLENGER, JOHN F., E. I. duPont de Nemours & Co., Wilmington, Del.
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*CHITTENDON, HOWARD L., Main St., Clinton, Conn.
*CHRISTENSEN, EINAR, 1600 Arch St., Philadelphia, Pa.
CHRISTOPHERSEN, O. E., Hires, Castner & Harris, Inc., Philadelphia, Pa.
*CHUBB, JOS. H., Universal Portland Cement Co., 210 S. LaSalle St., Chicago, Ill.
*CLAIR, MILES N., 5 Chester St., Newton Highlands, Mass.
*CLARK, FRANK E., Bridgeport Stone Co., Bridgeport, Conn.
*CLARK, W. KEITH, The Cincinnati Concrete Co., Cincinnati, Ohio.
*CLEMMER, H. F., 40 Rector St., New York, N. Y.
CLINTON, WM. H., Villanova, Pa.
*CLOUSING, LOUIS, 213 City Hall, Minneapolis, Minn.
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COE, FRANK C., Celite Products Co., Philadelphia, Pa.
*COFFMAN, HERBERT, Hercules Cement Corp., 1700 Walnut St., Philadelphia, Pa.
*COGHLAN, RAPIER R., S. W. Portland Cement Co., Pasadena, Calif.
*COLBURN, D. S., Marquette Cement Co., 140 S. Dearborn St., Chicago, Ill.
*COLLINS, D. R., 47th & State St., Milwaukee, Wis.
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*CONDON, JOHN, Turner Construction Co., 1700 Walnut St., Philadelphia, Pa.
*CONRADES, EDWIN H., St. Louis Material & Supply Co., St. Louis, Mo.
*CONWELL, E. L., 2024 Arch St., Philadelphia, Pa.
*COOKE, C. E., Kalman Floor Co., Wrigley Bldg., Chicago, Ill.
COURTNEY, N. C., Broad & Arch Sts., Philadelphia, Pa.
*CORRADO, ANTHONY, 58 Wainwright St., Providence, R. I.
CORSNON, S. K., Hartboro Concrete Products Co., Hartboro, Pa.
COTTER, MORTIMER T., Penn. Dixie Portland Cement Co., Philadelphia, Pa.
COTTON, C. C., Commercial Shearing & Stamping Co., Youngstown, Ohio.
COVELL, EARL D., 1609 W. Tioga St., Philadelphia, Pa.
*CRABBS, AUSTIN, The Cement Products Co., Davenport, Ia.
*CRANE, THEODORE, Winchester Hall, New Haven, Conn.
COYLE, JOHN J., Sinking Springs Concrete Co., Sinking Springs, Pa.
CREW, S. I., Crew Builders' Supply Co., Cincinnati, Ohio.
CROMWELL, WM. A., Camden Forge Co., West Collingswood, N. J.
CROSBY, E. S., Celite Products Co., 11 Broadway, New York, N. Y.
*CROZIER, ALFRED OWEN, 54 W. 40th St., New York, N. Y.
*CRUM, R. W., Iowa State Highway Comm., Ames, Ia.
*CRUME, WM. H., Crume Brick Co., Dayton, Ohio.

DAHL, LOUIS A., Lehigh Portland Cement Co., Allentown, Pa.
*DALCO, LOUIS A., Decorative Stone Co., New Haven, Conn.
DARBY, F. N., U. G. I. Contracting Co., Philadelphia, Pa.
*DAVIS, HERBERT A., Wash. Concrete Products Corp., Washington, D. C.
*DAVIS, RAYMOND E., University of California, Berkeley, Calif.
DAVIS, SPENCER H., Philadelphia Electric Co., Philadelphia, Pa.
*DAVISON, R. GLENN, Jamesburg, N. J.
DELEVANTE, C. A., Mohawk Stucco Co., Inc., Brooklyn, N. Y.
DEMCHICK, ISRAEL, 4631 N. Warnock St., Philadelphia, Pa.
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*DEVLIN, JOHN J., 100 E. 42nd St., New York, N. Y.
*DEVOS, A. W., Wm. H. Devos Co., Milwaukee, Wis.
*DISNEY, CHAS. P., New Union Station, Toronto, Ont., Canada.
*DOANE, LOUIS H., North Glenside, Pa.
*DOE, N. L., Turner Construction Co., Mt. Vernon, N. Y.
DONOVAN, G. WM., JR., E. I. duPont Co., 6209 Marsden St., Philadelphia, Pa.
*DOUGLASS, A. S., The Detroit Edison Co., Detroit, Mich.
DOUD, CLARENCE, Doud Concrete Products Co., Malone, N. Y.
*DOWNER, ROBERT G., Irish Hill Supply Co., 510 Wilson Bldg., Camden, N. J.
*DOWNS, C. S., Artstone Products, Inc., 52 Vanderbilt Ave., New York, N. Y.
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*DUNBAR, JAS. Y., Artstone Products, Inc., 25 E. 193rd St., New York, N. Y.
DUNCAN, CHESTER I., Wm. H. Gravell, Inc., Philadelphia, Pa.
DUNN, W. R., Vulcanite Portland Cement Co., Phillipsburg, N. Y.
*DUNNELLS, C. G., Century Bldg., Pittsburgh, Pa.
*DURGIN, F. L., Cramp & Co., 801 Denckla Bldg., Philadelphia, Pa.
*DUTTON, EARL S., 33 W. Tremlett St., Boston, Mass.
*EADES, HARRY H., National Steel Fabric Co., Pittsburgh, Pa.
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EDKINS, G. S., 1520 Locust St., Philadelphia, Pa.
EDMONDS, S. R., Edmonds' Art Stone Co., Washington, D. C.
*EGELHOFF, R. F., 11 Goodell St., Buffalo, N. Y.
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*FARMER, H. G., Frick Bldg., Pittsburgh, Pa.
*FERGUSON, LEWIS R., 342 Madison Ave., New York, N. Y.
*FERGUSON, M. W., Stone Tile & Supply Co., Roanoke, Va.
FERRER, RUSSELL R., Mastic Cement Block Works, Center Moriches, L. I.
*FINLAY, L. G., Raymond Concrete Pile Co., 140 Cedar St., New York, N. Y.
*FITZMAURICE, EDMUND J., Pennsylvania Bldg., Philadelphia, Pa.
*FLAM, STEPHEN, Supertile Machiery Corp., Tecumseh, Mich.
FLANAGAN, CHARLES A., Deputy Chief Bureau Bldg. Insp., Philadelphia, Pa.
*FOGG, RALPH J., Lehigh University, Bethlehem, Pa.
FOLLIN, JAMES W., Harrisburg, Pa.
*FORMIGLI, O. L., Formigli Arch. Stone Co., Philadelphia, Pa.
*FOSTER, ALEXANDER, Chas. Warner Co., 1518 Walnut St., Philadelphia, Pa.
*FOSTER, C. B., Foster Eng. Service Co., Indianapolis, Ind.

FOWLKES, JR., SAM., Philadelphia, Pa.
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*FRAUENFELDER, HERMAN, 4600 Chippewa St., St. Louis, Mo.
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FREEDMAN, LOUIS, 909 Fellsway, Medford, Mass.
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FRIEDGEN, C. F., U. G. I. Contracting Co., Broad and Arch Sts., Philadelphia, Pa.
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FULLER, M. O., 732 7th Ave., Bethlehem, Pa.
FINCK, J. C., 1089 Broadway, Albany, N. Y.
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FURTON, W. F., Furton Bros., 224 Bryant, Washington, D. C.
FURCH, MR., St. Louis Material & Supply Co., St. Louis, Mo.
GAGE, R. B., Box 106, Trenton, N. J.
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GARRIGAN, D. W., 216 Walnut St., Philadelphia, Pa.
*GASTON, H. F., 250 S. Broad St., Philadelphia, Pa.
*GEORGE, W. B., Straub Block Co., Warren, Ohio.
GEORGE, J. DUNCAN, Celite Products Co., 316 Bulletin Bldg., Philadelphia, Pa.
*GERNAND, R. N., Anti-Hydro Waterproofing Co., Newark, N. J.
GERWIG, M. ALBERT, The Ballinger Co., Glenside, Pa.
*GIBSON, JAMES E., Wood-Norton Apt., Germantown, Pa.
*GIDDINGS, WALTER S., Phila. Partition and Bldg. Block, Philadelphia, Pa.
*GIESECKE, F. E., College Station, Texas.
*GILMAN, CHAS., Massey Concrete Products Co., 50 Church St., New York, N. Y.
*GINSBERG, FRANK I., H. O. Penn Co., 18 E. 41st St., New York, N. Y.
*GLASHEEN, R. E., 1859 N. 9th St., Philadelphia, Pa.
*GLOSE, ROBERT L., Natl. Steel Fabric Co., Union Trust Bldg., Pittsburgh, Pa.
*GOLDBECK, A. T., Natl. Crushed Stone Assn., Washington, D. C.
GOOD, FRANK D., Chalfont, Pa.
GOODE, E. B., Hercules Cement Corp., 441 Lexington Ave., New York, N. Y.
*GORSUCH, V. D., Bessemer Cement Corp., Youngstown, Ohio.
GOSSMAN, A. L., 11 Broadway, New York, N. Y.
GRANGER, D. GEO., Stone & Webster, Inc., Boston, Mass.
*GRAVELL, WM. H., 225 S. 15th St., Philadelphia, Pa.
*GRAY, H. A., Park Sq. Bldg., Boston, Mass.
*GREENMAN, RUSSELL S., 10 Steuben St., Albany, N. Y.
*GRELIS, JOHN J., JR., Concrete Steel Co., 827 Dorset St., Philadelphia, Pa.
GRUBBS, N. S., 75 Branch St., Mt. Holly, N. J.
GRUBE, L. E. Sheboygan Brick Co., Sheboygan, Wis.
*HACKMAN, M. L., Lancaster Concrete Tile Co., 234 B, Water St., Lancaster, Pa.
HALEY, T. J., Dupont Eng. Co., 5549 Hazel Ave., Philadelphia, Pa.
HARDESTY, DICK, St. Thomas, Pa.
HARDESTY, H. W., Reinhart Concrete Block Co., Ft. Thomas, Ky.
HATCH, JOHN E., 4117 Walnut St., Philadelphia, Pa.
*HARDESTY, J. M., Bell Telephone Lab., 463 West St., New York, N. Y.
HARPER, CHAS. S., 253 Broadway, New York, N. Y.
*HART, W. E., Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.
*HARRINGTON, W. C., 131 Redfield Pl., Syracuse, N. Y.

HARRIS, A. C., Midland Supply & Coal Co., Alton, Ill.
*HARTER, B. D., Harter-Marblecrete Stone Co., Oklahoma City, Okla.
*HARTLESS, T. B., Froehling & Robertson, Richmond, Va.
HARVEY, G. L., JR., Philadelphia Electric Co., Philadelphia, Pa.
*HATT, W. K., Lafayette, Ind.
*HAY, W. W., New Brunswick, N. J.
*HEBOLD, DENIS, 217 W. Sparks St., Philadelphia, Pa.
*HECK, GEO. H., Natl. Cinder Products Co., Philadelphia, Pa.
HENDERSON, J. H., Granite Concrete Block Co., Ltd., Toronto, Canada.
*HENDRICK, THOS. K. A., 58 Forest Ave., New Rochelle, N. Y.
*HENRY, MURRAY H., Center Ave. and Dithridge St., Pittsburgh, Pa.
*HERZOG, C. J., 310 Magee Bldg., Pittsburgh, Pa.
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HESSELDER, L. G., 4800 Walnut St., Philadelphia, Pa.
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HILLMAN, CHAS. S., 400 Chestnut St., Philadelphia, Pa.
HIPP, GEO. E., Buffalo Litholite Co., Lackawanna, N. Y.
*HITCHCOCK, F. A., 5101 38th St. N. W., Washington, D. C.
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*HOLLISTER, S. C., Swarthmore, Pa.
*HOLM, W. M., 33 W. Grand Ave., Chicago, Ill.
*HORST, A. E., Henry W. Horst Co., Rock Island, Ill.
*HOWE, HENRY L., City Hall, Rochester, N. Y.
*HOWE, H. N., Memphis, Tenn.
HOWELL, RICHARD, Hollywood Bldg., Block Co., Bethlehem, Pa.
HOWELL, R. P., Fuller Co., Catasauqua, Pa.
*HUBBARD, FRED, 1200 City Bank Bldg., Youngstown, Ohio.
HUGHES, H. WALTER, City of Rochester, Rochester, N. Y.
*HUGHES, E. E., Franklin Steel Works, Franklin, Pa.
HULBERT, E. H., Driscoll Bros. & Co., Ithaca, N. Y.
HULME, NORMAN, 1524 Chestnut St., Philadelphia, Pa.
*HUMPHREY, D. S., Euclid Beach Park, Cleveland, Ohio.
HUMPHREY, H. B., 929 Chestnut St., Philadelphia, Pa.
*HUMPHREY, RICHARD L., Union League, Philadelphia, Pa.
*HUNT, PHIL., Hunt Stone-Tile Co., Salem, Mass.
HUTCHINS, CHAS. O., Wyoming Sand & Stone Co., Wilkes-Barre, Pa.
*HUTCHINS, L. A., Wolverine P. C. Co., Coldwater, Mich.
*HUTCHINSON, G. W., Lake Worth, Fla.
*INGBERG, S. H., Bureau of Standards, Washington, D. C.
*JACKSON, F. H., Bureau of Standards, Washington, D. C.
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JANSSON, MARTIN E., North Jersey Dist. Water Supply Co., Wanague, N. J.
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JENNINGS, WM. A., Southeastern Portland Cement Co., Osborn, Ohio.
JESTER, CHAS. H., 1317 Spruce St., Philadelphia, Pa.
*JOHNSON, A. V., Ideal Cement Stone Co., Omaha, Neb.
*JOHNSON, N. C., 342 Madison Ave., New York, N. Y.
*JOHNSON, R. C., Immel Constr. Co., Fond du Lac, Wis.
*JOHNSON, R. W., Concrete Reinforcing Steel Inst., Chicago, Ill.
*JOHNSON, VIRGIL L., 29 W. Upsal St., Philadelphia, Pa.
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JORDAN, WALTER L., 30 Church St., New York, N. Y.

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*KEATINGE, PAUL F., The Atlas Portland Cement Co., St. Louis, Mo.
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*KELLY, N. A., Universal Portland Cement Co., New York, N. Y.
*KENOGRATH, E., Penn.-Dixie Cement Corp., Chattanooga, Tenn.
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*KIRSHBAUM, HERMAN, 5816 Catharine St., Philadelphia, Pa.
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*KLEIN, W. H., Penn.-Dixie Cement Co., Chattanooga, Tenn.
*KLOTZ, PAUL R., N. Plymouth St., Allentown, Pa.
*KNEELAND, H. D., Calif. Stucco Prod. Co., Rochester, N. Y.
KNIGHT, A. W., Celite Products Co., Glendale, Calif.
KNOPEL, HERBERT J., Locust and 15th Sts., Philadelphia, Pa.
KNOWLTON, WINFIELD B., Amer. Woolen Co., Andover, Mass.
KOCH, CHAS. W., York Lithoid Products Co., York, Pa.
KOELLE, WM. F., JR., 6424 City Line, Philadelphia, Pa.
KRATZ, JOS. S., Alpha Portland Cement Co., 427 S. West End Ave., Lancaster, Pa.
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KRIEG, WILLIAM E., Calif. Stucco Prod. Co. of N. J., East Orange, N. J.
KUEGLE, PAUL C., The Youngstown Sheet & Tube Co., Youngstown, Ohio.
*KREMER, ERNEST, Wright & Kremer, Niagara Falls, N. Y.
KURTZ, JOHN A., Ephrata, Pa.
KURTZ, J. KREIDER, Ephrata, Pa.
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LEVINSOHN, SAM., Vitrostone Mfg. Co., Baltimore, Md.
*LEVISON, ARTHUR A., Blaw-Knox Co., Pittsburgh, Pa.
LEWIN, JOHN C., Dow Chemical Co., Midland, Mich.
*LEWIS, E. W. REED, 1754 Buhl Bldg., Detroit, Mich.
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LIND, FRANK A., Altoona Duntile Products Co., Altoona, Pa.
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LINDSAY, G. L., Universal Portland Cement Co., 210 S. LaSalle St., Chicago, Ill.
*LINDSLEY, C. E., C. E. Lindsley Co., Irvington, N. J.
*LIVINGSTON, J. A., Granite Concrete Block Co., Ltd., Toronto, Ont., Canada.
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*LOCKHARDT, W. F., 347 Madison Ave., New York, N. Y.
*LONG, JAMES B., Norristown, Pa.

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LUMMIS, CLARENCE, 522 Stevens St., Camden, N. J.
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*MCCLURE, D. A., Jamestown Block & Tile Co., Jamestown, N. Y.
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*MACKAY, J. R., The Decorative Stone Co., New Haven, Conn.
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*MCCANLESS, W. F., Ornamental Stone Co., Charlotte, N. C.
*MCCOMAS, WM. E., Portland Cement Assn., Philadelphia, Pa.
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*MCCULLOUGH, F. M., Carnegie Inst. of Technology, Pittsburgh, Pa.
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*MCKENZIE, C. L., Duquesne Slag Products Co., Pittsburgh, Pa.
*MCMILLAN, F. R., Portland Cement Assn., Chicago, Ill.
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INDEX